Technical Report Documentation Page

1. REPORT No.
643230

2. GOVERNMENT ACCESSION No.

3. RECIPIENT'S CATALOG No.

4. TITLE AND SUBTITLE
Experimental Asphalt Test Sections On The Newcastle And Colfax Jobs

5. REPORT DATE
February 1968

6. PERFORMING ORGANIZATION

7. AUTHOR(S)
Zube, E., and Skog, J.B.

8. PERFORMING ORGANIZATION REPORT No.
643230

9. PERFORMING ORGANIZATION NAME AND ADDRESS
State of California
Department of Public Works
Division of Highways
Materials and Research Department

10. WORK UNIT No.

11. CONTRACT OR GRANT No.

12. SPONSORING AGENCY NAME AND ADDRESS

13. TYPE OF REPORT & PERIOD COVERED
Final Report

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

16. ABSTRACT
Test sections were placed during construction in 1959 of the asphalt concrete pavement on Contracts 57-3TC27, the Newcastle job and 57-3TC21, the Colfax job. the primary purpose of the project was to study the relation between various kinds of compaction and density and permeability for different mix designs. On completion of these studies during construction, it was decided to also study the service performance of the binder in the different sections.

The Newcastle project was paved with an 85-100 grade paving asphalt from a single source. The various test sections were as follows. Two of the sections were controls and contained a standard Type A paving mixture. The other sections were a Type A mix with 2.3% cement added as a filler dust, a Type A mix with 2.0% limestone and a Type A mix where the attempt was made to remove all of the -200 material. This was known as the "No Dust Section".

The Colfax project was paved with an 85-100 grade paving asphalt from a single source. On this project two test sections were placed with a Type A paving mixture. One section contained 4.5% asphalt, the other 5.5% asphalt.

At various service life periods, deflection measurements, pavement performance and properties of recovered asphalt have been determined. The cracking pattern on the Newcastle job has all been transverse or longitudinal and no fatigue cracking was found up to 104 months service life. Virtually no cracking has occurred on the Colfax job. Average deflection results on both jobs have been uniformly low. No relation was found on the Newcastle job between the amount of cracking in any one section and the recovered properties of the asphalt. However, the hardening of the asphalt was found to be related to the original void content and the rate of decrease in void content during service life.

17. KEYWORDS
Pavements, asphaltic concrete, fillers, test sections, cement, limestone filler, pavement deflections, pavement performance, cracking, mix design, compaction, density

18. No. OF PAGES:
88

19. DRI WEBSITE LINK
http://www.dot.ca.gov/hq/research/researchreports/1968/68-23.pdf

20. FILE NAME
68-23.pdf

HIGHWAY RESEARCH REPORT

EXPERIMENTAL ASPHALT TEST SECTIONS ON THE NEWCASTLE AND COLFAX JOBS

FINAL REPORT

February, 1968

68-23

STATE OF CALIFORNIA

TRANSPORTATION AGENCY

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

MATERIALS AND RESEARCH DEPARTMENT

RESEARCH REPORT

NO. M & R 643230

State of California
Department of Public Works
Division of Highways
Materials and Research Department

February 1968
Final Report
M & R No. 643230

Mr. J. A. Legarra
State Highway Engineer

Dear Sir:

Submitted herewith is a research report entitled:

EXPERIMENTAL ASPHALT TEST
SECTIONS ON THE
NEWCASTLE AND COLFAX
JOBS

ERNEST ZUBE
Principal Investigator

JOHN SKOG
Co-Investigator

Very truly yours,

JOHN L. BEATON
Materials and Research Engineer

REFERENCE: Zube, E., Skog, J. B., "Experimental Asphalt Test Sections on the Newcastle and Colfax Jobs", State of California, Department of Public Works, Division of Highways, Materials and Research Department, Research Report 643230, February 1968.

ABSTRACT: Test sections were placed during construction in 1959 of the asphalt concrete pavement on Contracts 57-3TC27, the Newcastle job and 57-3TC21, the Colfax job. The primary purpose of the project was to study the relation between various kinds of compaction and density and permeability for different mix designs. On completion of these studies during construction, it was decided to also study the service performance of the binder in the different sections.

The Newcastle project was paved with an 85-100 grade paving asphalt from a single source. The various test sections were as follows. Two of the sections were controls and contained a standard Type A paving mixture. The other sections were a Type A mix with 2.3% cement added as a filler dust, a Type A mix with 2.0% limestone and a Type A mix where the attempt was made to remove all of the -200 material. This was known as the "No Dust Section".

The Colfax project was paved with an 85-100 grade paving asphalt from a single source. On this project two test sections were placed with a Type A paving mixture. One section contained 4.5% asphalt, the other 5.5% asphalt.

At various service life periods, deflection measurements, pavement performance and properties of recovered asphalt have been determined. The cracking pattern on the Newcastle job has all been transverse or longitudinal and no fatigue cracking was found up to 104 months service life. Virtually no cracking has occurred on the Colfax job. Average deflection results on both jobs have been uniformly low. No relation was found on the Newcastle job between the amount of cracking in any one section and the recovered properties of the asphalt. However, the hardening of the asphalt was found to be related to the original void content and the rate of decrease in void content during service life.

KEY WORDS: Pavements, asphaltic concrete, fillers, test sections, cement, limestone filler, pavement deflections, pavement performance, cracking, mix design, compaction, density.

CONTENTS

	Page
INTRODUCTION	1
CONCLUSIONS	2
DESCRIPTION OF TEST SECTIONS	3
FIELD AND LABORATORY TESTING PROGRAM	4
TEST RESULTS	5
DISCUSSION	5-6-7
REFERENCES	8
TABLES	
FIGURES	
APPENDIX	

INTRODUCTION

During the 1957-58 and 59 construction seasons extensive compaction studies were performed on a series of asphalt concrete paving jobs. Two of these contracts were 57-3TC21, Road 03-Pla-37-B and 57-3TC27, Road 03-Pla-17,37-B, Aub. Contract 57-3TC21 is also known as the Colfax job and 57-3TC27 is known as the Newcastle job.

Although the primary purpose of the project was to study the relation between various forms of compaction and permeability for different mix designs, it was also decided to study the performance of the binder in the different sections during service life. Previous studies relative to these two contracts are reported in references 1 through 5.

The purpose of this final report is to present the evaluation of the pavement together with change in binder properties during a service life of nine years.

CONCLUSIONS

The asphalt content was found to be the most important factor in the reduction of permeability of asphalt concrete by construction compaction.

The amount of cracking in the Newcastle test sections could not be related to the deflections or properties of the recovered asphalt. The cracking on this job was transverse and longitudinal, and no evidence of fatigue type cracking was found up to 104 months of service life. It appears that the cracking is mainly of the reflection type from the underlying cement treated base.

The percentage of voids immediately after construction and the rate of change of void content during the initial period of service life are very important factors in the control of the weathering rate of the binder.

DESCRIPTION OF TEST SECTIONS

The pavement section for Contract 57-3TC27, (Newcastle job) was composed of 0.25' Type A, 3/4" Max. Asphalt Concrete. The test sections for this contract were each approximately seven hundred feet long and were composed of the following:

Section No.	Asphalt Grade & Source	Asphalt Content	Description
A	85-100 Grade Douglas Oil Santa Maria	5.2	Standard Type A Paving Mixture - Contract Rolling - Control Section - Sta.220+00-226+70, WB Lanes.
B	" " "	5.2	Standard Type A Paving Mixture - Special Rolling - Sta.241+25-249+00,WB Lanes.
C	" " "	5.7	"No Dust" Paving Mixture - Special Rolling - Sta.234+00-241+25,WB Lanes.
D	" " "	4.5	Standard Type A +2.3% Cement Paving Mixture - Special Rolling - Sta.226+70-234+00,WB Lanes.
E	" " "	4.5	Standard Type A +2.0% Limestone Paving Mixture - Special Rolling - Sta.242+00-249+00,EB Lanes.

The pavement section for Contract 57-3TC21, (Colfax job) was composed of 0.25' Type A, 3/4" Max. Asphalt Concrete. The test sections for this contract were composed of the following:

Section No.	Asphalt Grade & Source	Asphalt Content	Description
1	85-100 Grade Union Oil Co. Oleum	4.5	Standard Type A Paving Mixture.
2	" " "	5.5	Standard Type A Paving Mixture.

FIELD AND LABORATORY TESTING PROGRAM

Periodical pavement condition and deflection surveys have been performed on both contracts. Cores have also been removed at intervals for studies on the changes in pavement and binder properties. Tests on the cores have included stability, cohesion, extraction, gravity, and air permeability tests.

Various tests were performed on the recovered asphalt from the entire core. These included penetration, softening point and ductility tests and viscosity and microductility tests. Also certain cores were sliced, and viscosities determined for hardening at various depths.

TEST RESULTS

Pavement deflection results were not obtained in 1967 for the Colfax job because previous runs have consistently been very low, and the present pavement continues to be in excellent condition in terms of cracking.

The summary of cracking on the Newcastle Test Sections is shown on Table A for observations performed 12, 24, 48 and 104 months after construction. Table B presents the average deflection readings for various periods of service life. Table C shows previous average deflections for the Colfax job. Appendix A contains tables showing all test results performed on cores obtained in January 1967 at the conclusion of service life studies. Previous core results are found in Reference 6. Tables D and E show the average properties of the recovered asphalt for various service periods on the two jobs. Tables F, G, H, and I present the average properties of the pavement cores for various service periods on the two jobs.

DISCUSSION

The cracking pattern of all five test sections on the Newcastle job has been similar during service life. The typical cracking pattern for all sections is shown in Fig. 1. Fatigue cracking has not been a problem on this job, and the initial transverse cracking may be assigned to reflection cracking from the cement treated base. As shown in Table A the first longitudinal cracking occurred during the 24-48 month period and continued to increase thereafter. The increase in total cracking, (transverse and longitudinal) during service life for the Newcastle test sections is shown in Fig. 2. There is a definite difference in the amount of cracking, Sections C and E, the "No Dust" and 2.0% Limestone Sections being much lower than the Standard Paving Mixture used in Sections A and B. The rate of increase in total cracking is very interesting. There is a rapid increase in transverse cracking up to 12 months and virtually no increase up to 24 months. This is all transverse cracking since no longitudinal cracking was observed during the first 24 months of service life. One may conclude, based on the very low deflection results during this period, that the transverse cracking was of the reflection type and gradually came to virtually a halt during the 12-24 month period. However, thereafter, in the 24-48 month period the transverse cracking again increased at a rapid rate as shown in Fig. 3. This increase coupled with the start of longitudinal cracking increased the rate of total cracking to a high figure. Again in the 48-104 month periods the total cracking although increasing definitely slowed in comparison to the previous period.

All but one of the test sections were placed in the WB travel lane and adjacent to each other. The exception was the Limestone Section E which was located in the EB travel lane. Also the standard Type A Section A and B which have very nearly the same amount of cracking were at opposite ends of the test area and were separated by Sections C and D. This leads to the assumption that the type of mix may have been an important factor in the difference in total cracking beginning after 24 months of service. A comparison of the total cracking in the various sections with mix and recovered asphalt properties at 24 and 104 months of service life is shown below.

Section No.	Type of Mix	Age Mo	Total Cracking Ft/100Ft	Ave. Defl. 0.001"	Ave. Pen.	Ave. Shear Index
C	"No Dust"	23	48	9	28	0.09
		104	140	10	24	0.27
E	Type A +2% Limestone	23	43	7	20	0.22
		104	143	10	19	0.29
D	Type A + 2.3% Cement	23	52	11	19	0.31
		104	242	15	16	0.37
A	Type A Control	23	47	10	32	0.19
		104	330	16	29	0.20
B	Type A Control	23	54	11	29	0.15
		104	343	11	30	0.19

There is no relation between the amount of cracking and either the deflection or recovered asphalt properties. There is a possibility of explaining the rapid increase in cracking by an increase in shear index, but this did not prove to be the case, since the asphalt in Section D has a high shear index, but less cracking than Sections A and B which have definitely lower shear susceptibilities. It is concluded that the continued cracking after 24 months was mainly caused by reflection cracking from the cement treated base.

The Newcastle and Colfax jobs have also provided further evidence on the void-asphalt weathering relationship. This relation for the two jobs is shown in Table J. A study of these results indicates that the amount of voids shortly after compaction, together with the rate of change in voids during service life are primary factors in retention of penetration. The asphalt content is a very important factor not only during compaction, but also in the void reduction rate during service life. In any case the importance of attaining the lowest possible void content shortly after construction is shown by the remarkable retention of penetration in the 5.5% test section on the Colfax job. Although this section has not shown evidence of instability there was some bleeding during the first three years of service life. Therefore, we would not recommend mixes having such marked reduction in void content. The importance of careful control on the void content during design and construction, and the development of a method of predicting void content changes during service life appear to be important factors in slowing down the rate of weathering of the binder.

Hardening with depth curves on both jobs are shown in Figs. 4 and 5. It is very interesting to note that normal hardening with depth curves are found in the high asphalt sections on both jobs. The differences between the two normal curves and the abnormal bottom hardening in the other sections are definitely significant in that all the test sections were placed under the same conditions and checks on the cores shortly after construction indicated equivalent viscosity from top to bottom. The differences in hardening with depth on the Newcastle test section is not related to the amount of cracking. Sections C and E having the lowest amount of cracking exhibit opposite trends in hardening with depth. We presently believe that a reversal of the normal pattern of hardening is most significant in the case of fatigue cracking where the pavement has the greatest strain at the bottom. This form of cracking was not found on either the Newcastle or Colfax jobs. On the basis of these observations the conclusion is reached that reflection or temperature induced cracking is not influenced by the weathering profile of the binder within the pavement.

REFERENCES

1. "Compaction Studies of Asphalt Concrete Pavement as Related to the Water Permeability Test", E. Zube Highway Research Board, January 1962.

2. "Permeability Test", E. Zube, California Highways and Public Works, July, August 1963.

3. "Progress Report #1 on the Permeability Studies of Plant Mixed Surfacing and Related Problems". September 1957, Pavement Section.

4. "Progress Report #3 on the Permeability Studies of Plant Mixed Surfacing and Related Problems". January 1960, Pavement Section.

5. "Progress Report #4 on the Permeability Studies of Plant Mixed Surfacing and Related Problems". February 1960, Pavement Section.

6. Progress Report on "Field and Laboratory Studies of the Pavement Test Sections on Contracts 57-3TC21, Road III-Pla-37-B and 57-3TC27, Road III-Pla-17,37-B Aub." March 1961, Pavement Section.

TABLE A

Summary of Cracking of Asphalt Concrete
Pavement on Contract 57-3TC27, Road III-Pla-17,37-B, Aub.
(Newcastle Job)

Section No.	Description	Survey Date	Age Mo.	Travel Lane			Passing Lane			Total Travel and Passing
				Trans. Ft./100Ft.	Long. Ft./100Ft.	Alligator Cracking %	Trans. Ft./100Ft.	Long. Ft./100Ft.	Alligator Cracking %	
A	Standard Type A	May 1959	12	25	0	0	14	0	0	39
	Normal Rolling	May 1960	24	28	0	0	22	0	0	47
	Sta.220+00-226+70 Length = 670'	May 1962	48	62	112	0	53	27	0	254
	W.B. Lanes	Jan. 1967	104	63	174	0	53	40	0	330
B	Standard Type A	May 1959	12	20	0	0	19	0	0	39
	Special Rolling	May 1960	24	26	0	0	22	6	0	54
	Sta.241+25-249+00 Length = 775'	May 1962	48	36	84	0	36	49	0	205
	W.B. Lanes	Jan. 1967	104	44	185	0	44	70	0	343
C	"No Dust" Special Rolling	May 1959	12	21	0	0	20	0	0	41
	Sta.234+00-241+25 Length = 725'	May 1960	24	26	0	0	22	0	0	48
	W.B. Lanes	May 1962	48	38	23	0	37	9	0	107
		Jan. 1967	104	43	37	0	43	17	0	140

TABLE A - Continued

Section No.	Description	Survey Date	Age Mo.	Travel Lane			Passing Lane			Total Travel and Passing
				Trans. Ft./100Ft.	Long. Ft./100Ft.	Alligator Cracking %	Trans. Ft./100Ft.	Long. Ft./100Ft.	Alligator Cracking %	
D	Standard Type A	May 1959	12	23	0	0	27	0	0	50
	+2.3% Cement	May 1960	24	23	0	0	29	0	0	52
	Special Rolling Sta.226+70-234+00	May 1962	48	47	38	0	52	20	0	157
	Length = 630' W.B. Lanes	Jan. 1967	104	54	102	.01	57	29	0	242
E	Standard Type A	May 1959	12	17	0	0	19	0	0	36
	+2.0% Limestone	May 1960	24	22	0	0	21	0	0	43
	Special Rolling Sta.242+00-249+00	May 1962	48	47	7	0	38	5	0	97
	Length = 700' E.B. Lanes	Jan. 1967	104	54	17	0	51	21	0	143

TABLE B

Average Deflection Readings in 0.001" With 15,000 lbs. Axle Load

Contract 57-3TC27
Road III-Pla-17,37-B, Aub.
(Newcastle Job)

Date	Age Mo.	Section A Stand.Type A Normal Rolling W.B. Travel		Section B Stand.Type A Spec.Rolling W.B. Travel		Section C "No Dust" W.B. Travel		Section D Stand.Type A +2.3% Cement W.B. Travel		Section E Stand. Type A +2.0% Limestone W.B. Travel		Total Average All Sections Travel Lanes	
		OWT	IWT	OWT	IWT	OWT	IWT	OWT	IWT	OWT	IWT	OWT	IWT
5-28 58	After Const.	11	7	7	6	8	5	10	8	7	7	8	6
10-31 58	5	7	6	7	6	6	6	9	8	5	6	7	6
3-8 60	21.5	10	6	11	7	9	6	11	6	7	6	10	6
1-6 67	104	16	-	11	-	10	-	15	-	10	-	12	-

TABLE C

Average Deflection Readings in 0.001"

With 15,000 pounds Axle Load

Contract 57-3TC21

Road 03-Pla-37-B (Colfax Job)

Date	Age Mo.	Ave. Deflection E.B. Travel Lane, 4.5 & 5.5% Asphalt Sections	
		IWT	OWT
5-28-58	5.5	5	4
10-30-58	10.5	3	3
3-8-60	26.5	6	4
1-3-67	108	-	-

TABLE D

Physical Properties of Recovered Asphalt

Contract 57-3TC27
Road 03-Pla-17,37-B,Aub. (Newcastle Job)

Section No.	Description	Pvt. Age Mo.	Test Results on Recovered Asphalt					
			Pen. 77°F	S.P. °F	Stand. Duct. 77°F cm	Viscosity $5x10^{-2}$S.R. 77°F Megapoises	Shear Index	Micro Duct. 77°F mm
A	Standard Type A Normal Rolling Sta. 220+00-226+70 W. B. Lanes	23	32	139	64	12	0.19	-
		104	29	139	79-100+	20	0.20	14
B	Standard Type A Special Rolling Sta.241+25-249+00 W. B. Lanes	23	29	137	83	15	0.15	-
		104	30	139	22-100+	17	0.19	24
C	"No Dust" Special Rolling Sta234+00-241+25 W. B. Lanes	23	28	140	45	17	0.09	-
		104	24	148	45	24	0.27	8
D	Standard Type A +2.3% Cement Special Rolling Sta226+70-234+00 W. B. Lanes	23	19	149	9	26	0.31	-
		104	16	161	6	76	0.37	1
E	Standard Type A +2.0% Limestone Special Rolling Sta242+00-249+00 E. B. Lanes	23	20	143	12	22	0.22	-
		104	19	147	25	43	0.29	6

TABLE E

Physical Properties of Recovered Asphalt

Contract 57-3TC21
Road 03-Pla-37-B (Colfax Job)

Section No.	Description	Pvt. Age Mo.	Test Results on Recovered Asphalt					
			Pen. 77°F	S.P. °F	Stand. Duct. 77°F cm	Viscosity 5×10^{-2} S.R. 77°F Megapoises	Shear Index	Micro Duct. 77°F mm
1	Standard Type A 4.5% Asphalt	28	19	137	100+	21	0.09	-
		108	13	142	54	95	0.17	1
2	Standard Type A 5.5% Asphalt	28	41	125	100+	4.4	0.02	-
		108	45	133	100+	7.4	0.0	92

TABLE F

Summary of Average Physical Test Results on Pavement Samples

Contract 57-3TC27, Road 03-Pla-17,37-B,Aub.

Pave. Age Mo.	Mix Type	Sp. Grav. Wax	Ave. Stab. 140°F	Ave. Coh. 140°F	Field Asph. %	% Asph. Ext.	Grading									
							3/4	1/2	3/8	4	8	16	30	50	100	200
Orig. Field Mix	Standard Type A Normal Rolling	2.35	33	280	5.2	4.5	99	78	60	41	31	26	20	11	5	3
23		2.38	33	300	-	4.9	100	79	64	45	35	29	23	13	7	4
104		2.40	-	-	-	5.1	100	85	69	48	37	31	24	13	8	5
Orig. Field Mix	Standard Type A Special Rolling	2.35	33	280	5.2	4.5	99	78	60	41	31	26	20	11	5	3
23		2.38	32	292	-	5.1	99	82	70	49	40	31	25	15	8	4
104		2.39	-	-	-	4.9	100	84	69	46	36	30	24	13	7	5
Orig. Field Mix	"No Dust"	2.35	33	245	5.7	4.9	100	84	69	49	39	31	23	10	4	3
23		2.33	30	139	-	5.2	99	81	67	51	40	33	24	11	6	4
104		2.37	-	-	-	5.3	100	80	65	49	38	32	24	11	6	4
Orig. Field Mix	Standard Type A +2.3% Cement	2.34	34	110	4.5	4.3	98	86	74	55	43	35	27	12	6	5
23		2.30	29	150	-	4.3	99	89	72	52	40	33	25	12	7	4
104		2.33	-	-	-	4.2	100	82	66	48	38	31	24	12	6	4

TABLE F (CON'T)

Summary of Average Physical Test Results on Pavement Samples

Contract 57-3TC27, Road 03-Pla-17,37-B,Aub.

Pave. Age Mo.	Mix Type	Sp. Grav. Wax	Ave. Stab. 140°F	Ave. Coh. 140°F	Field Asph. %	% Asph. Ext.	Grading									
							3/4	1/2	3/8	4	8	16	30	50	100	200
Orig. Field Mix	Standard Type A +2.0% Limestone Dust	2.33	31	110	4.5	4.1	99	79	65	47	37	30	24	11	6	4
23		2.32	31	146	-	4.0	99	81	67	50	38	32	25	13	7	5
104		2.35	-	-	-	4.2	100	81	67	47	38	31	24	12	7	5

TABLE G

Summary of Average Physical Test Results on Pavement Samples

Contract 57-3TC21 Road 03-Pla-37-B (Colfax Job)

Pave. Age Mo.	Mix Type	Sp. Grav. Wax	Ave. Stab. 140°F	Ave. Coh. 140°F	Field Asph. %	% Asph. Ext.	Grading									
							3/4	1/2	3/8	4	8	16	30	50	100	200
Orig. Field Mix	Standard Type A 4.5% Asphalt	2.33	36	100	4.5	4.5	100	84	68	48	36	27	19	11	7	5
28		2.31	28	199	"	4.2	100	90	69	47	35	27	20	11	7	5
108		2.32	-	-	"	4.3	100	84	68	46	35	27	19	11	7	5
Orig. Field Mix	Standard Type A 5.5% Asphalt	2.34	34	133	5.5	5.4	100	83	67	46	36	27	18	10	7	5
28		2.39	28	162	"	5.2	100	86	66	44	34	26	19	11	7	5
108		2.41	-	-	"	5.3	100	84	65	44	33	26	18	10	7	5

TABLE H

Summary of Average Properties of Pavement Samples

Contract 57-3TC27 Road 03-Pla-17,37-B,Aub.

Pvt. Age Mo.	Mix Type		Theo. Max. Density	Ave. Spec. Grav. (Wax)	Ave. Rel. Density	Ave. % Voids	Ave. % Rel. Comp.	Wt. per cu.ft.	% Asphalt Field	% Asphalt Lab. Ext.	Ave. Stab. 140°F	Ave. Coh. 140°F
Orig Mix	Standard Type A Normal Rolling Section A	District	2.42	2.36	97.6	2.4	-	147	5.2	5.0	40	-
		Sacto.	2.42	2.35	97.1	2.9	100	147	5.2	4.5	33	280
1			-	2.31	95.4	4.6	98.2	144	-	-	29	190
23			-	2.38	98.3	1.7	101.3	149	-	4.9	33	300
104			-	2.40	98.9	1.1	102.2	150	-	5.1	-	-
Orig Mix	Standard Type A Special Rolling Section B		2.42	2.35	97.1	2.9	100	147	5.2	4.5	33	280
1			-	2.32	95.9	4.1	98.7	145	-	-	29	171
23			-	2.38	98.3	1.7	101.3	149	-	5.1	32	292
104			-	2.39	98.6	1.4	101.7	149	-	4.9	-	-
Orig Mix	"No Dust" Section C		2.41	2.35	97.4	2.6	100	147	5.7	4.9	33	245
1			-	2.28	94.6	5.4	97.0	142	-	-	27	94
23			-	2.33	96.7	3.3	99.1	145	-	5.2	30	139
104			-	2.37	98.3	1.7	100.8	148	-	5.3	-	-

TABLE H (CON'T)

Summary of Average Properties of Pavement Samples

Contract 57-3TC27 Road 03-Pla-17,37-B,Aub.

Pvt. Age Mo.	Mix Type	Theo. Max. Density	Ave. Spec. Grav. (Wax)	Ave. Rel. Density	Ave. % Voids	Ave. % Rel. Comp.	Wt. per cu.ft.	% Asphalt		Ave. Stab. 140°F	Ave. Coh. 140°F
								Field	Lab. Ext.		
Orig Mix	Standard Type A +2.3% Cement Section D	2.45	2.34	95.4	4.6	100	146	4.5	4.3	34	110
1		-	2.28	93.0	7.0	97.4	142	-	-	25	145
23		-	2.30	93.9	6.1	98.3	144	-	4.3	29	150
104		-	2.33	94.9	5.1	99.6	145	-	4.2	-	-
Orig Mix	Standard Type A + 2.0% Limestone Dust Section E	2.45	2.33	95.1	4.9	100	145	4.5	4.1	31	110
1		-	2.28	93.0	7.0	97.9	142	-	-	29	96
23		-	2.32	94.6	5.4	99.6	145	-	4.0	31	146
104		-	2.35	95.9	4.1	100.8	147	-	4.2	-	-

TABLE I

Summary of Average Properties of Pavement Samples

Contract 57-3TC21 Road 03-Pla-37-B (Colfax Job)

Pvt. Age Mo.	Mix Type	Theo. Max. Density	Ave. Spec. Grav. (Wax)	Ave. Rel. Density	Ave. % Voids	Ave. % Rel. Comp.	Wt. per cu.ft.	% Asphalt		Ave. Stab. 140°F	Ave. Coh. 140°F
								Field	Lab. Ext.		
Orig Mix	Standard Type A 4.5% Asphalt	2.45	2.33	95.1	4.9	100	145	4.5	4.5	36	100
1		"	2.22	90.6	9.4	95	139	"	-	17	66
28		"	2.31	94.3	5.7	99	144	"	4.2	28	199
108		"	2.32	94.7	5.3	99	145	"	4.3	-	-
Orig Mix	Standard Type A 5.5% Asphalt	2.42	2.34	96.6	3.4	100	146	5.5	5.4	34	133
1		"	2.24	92.6	7.4	96	140	"	-	17	71
28		"	2.39	98.8	1.2	102	149	"	5.2	28	162
108		"	2.41	99.6	0.4	103	150	"	5.3	-	-

TABLE J

Relation Between Void and Penetration Change
During Service Life
Contract 57-3TC27
Road III-Pla-17,37-B, Aub.

Section No.	Description	Asphalt Content	Ave. Percent Voids			Ave. Rec. Pen.		
			1 Mo. Cores	23 Mo. Cores	104 Mo. Cores	Const.	23 Mo. Cores	104 Mo. Cores
A	Standard Type A Normal Rolling Sta. 220+00-226+70 W.B. Lanes	5.2	4.6	1.7	1.1	50	32	29
B	Standard Type A Special Rolling Sta.241+25-249+00 W.B. Lanes	5.2	4.1	1.7	1.4	50	29	30
C	"No Dust" Special Rolling Sta.234+00-241+25 W.B. Lanes	5.7	5.4	3.3	1.7	49	28	24
D	Standard Type A + 2.3% Cement Special Rolling Sta.226+70-234+00 W.B. Lanes	4.5	7.0	6.1	5.1	50	19	16
E	Standard Type A + 2.0% Limestone Special Rolling Sta.242+00-249+00 E.B. Lanes	4.5	7.0	5.4	4.1	48	20	19

Asphalt = 85-100 Grade - Douglas Oil Co., Santa Maria

TABLE J - Continued

Contract 57-3TC21
Road III-Pla-37-13

Section No.	Description	Asphalt Content	Ave. Percent Voids			Ave. Rec. Pen.		
			1 Mo. Cores	28 Mo. Cores	108 Mo. Cores	Const.	28 Mo. Cores	108 Mo. Cores
1	Standard Type A	4.5	9.4	5.7	5.3	52	19	13
2	Standard Type A	5.5	7.4	1.2	0.4	52	41	45

Asphalt = 85-100 Grade - Union Oil Co., Oleum

Figure 1

CRACKING PATTERN FOR SECTION A

CONT. 57-3TC27, ROAD 03-PLA-17, 37-B, AUB.
NEWCASTLE JOB

THE PATTERN IS TYPICAL FOR
THE OTHER FOUR TEST SECTIONS

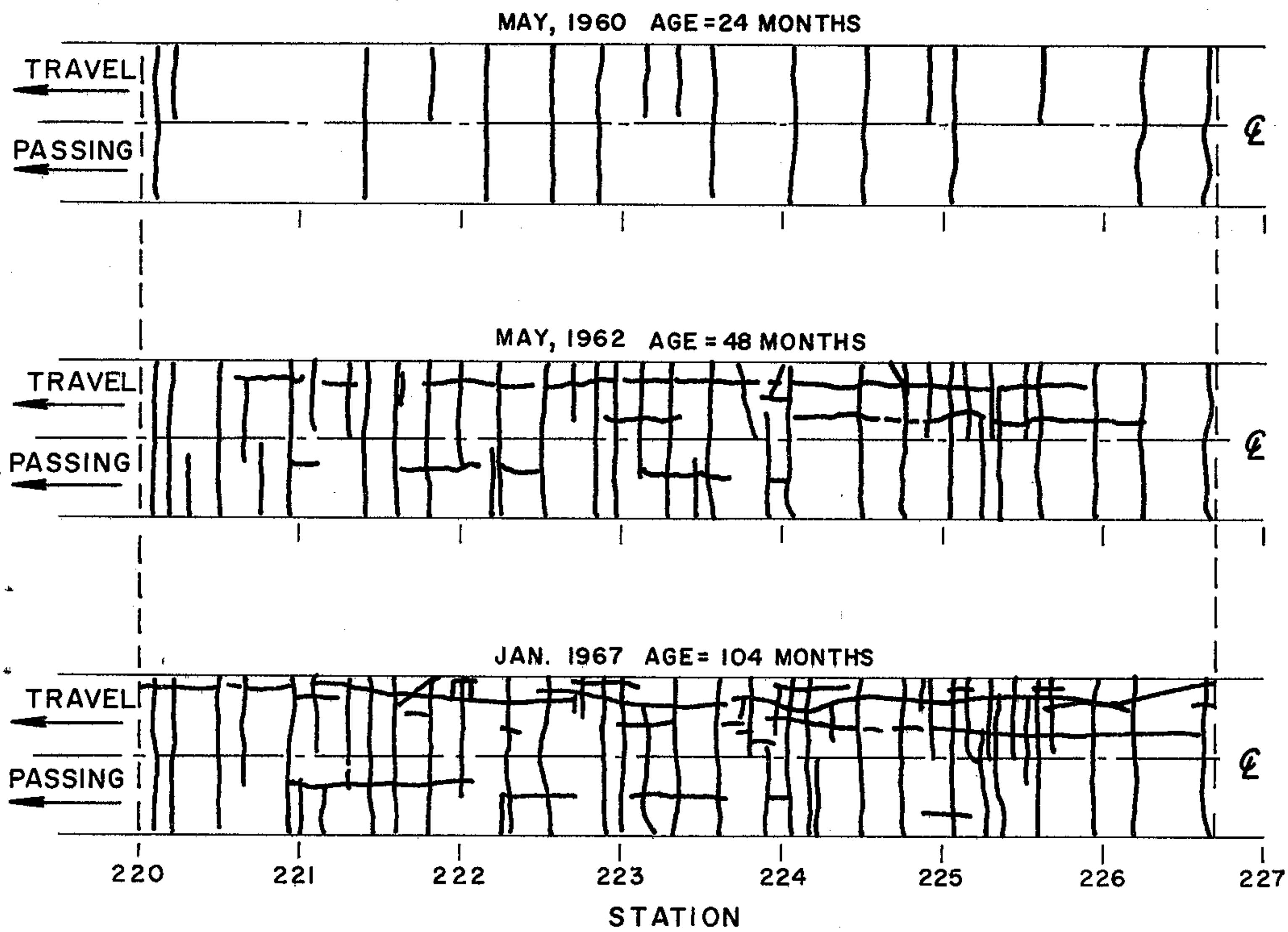


Figure 2

INCREASE IN TOTAL CRACKING OF TRAVEL AND PASSING LANES DURING SERVICE LIFE

CONT. 57-3TC27 ROAD 03-Pla-17, 37-B, AUB.

NEWCASTLE JOB

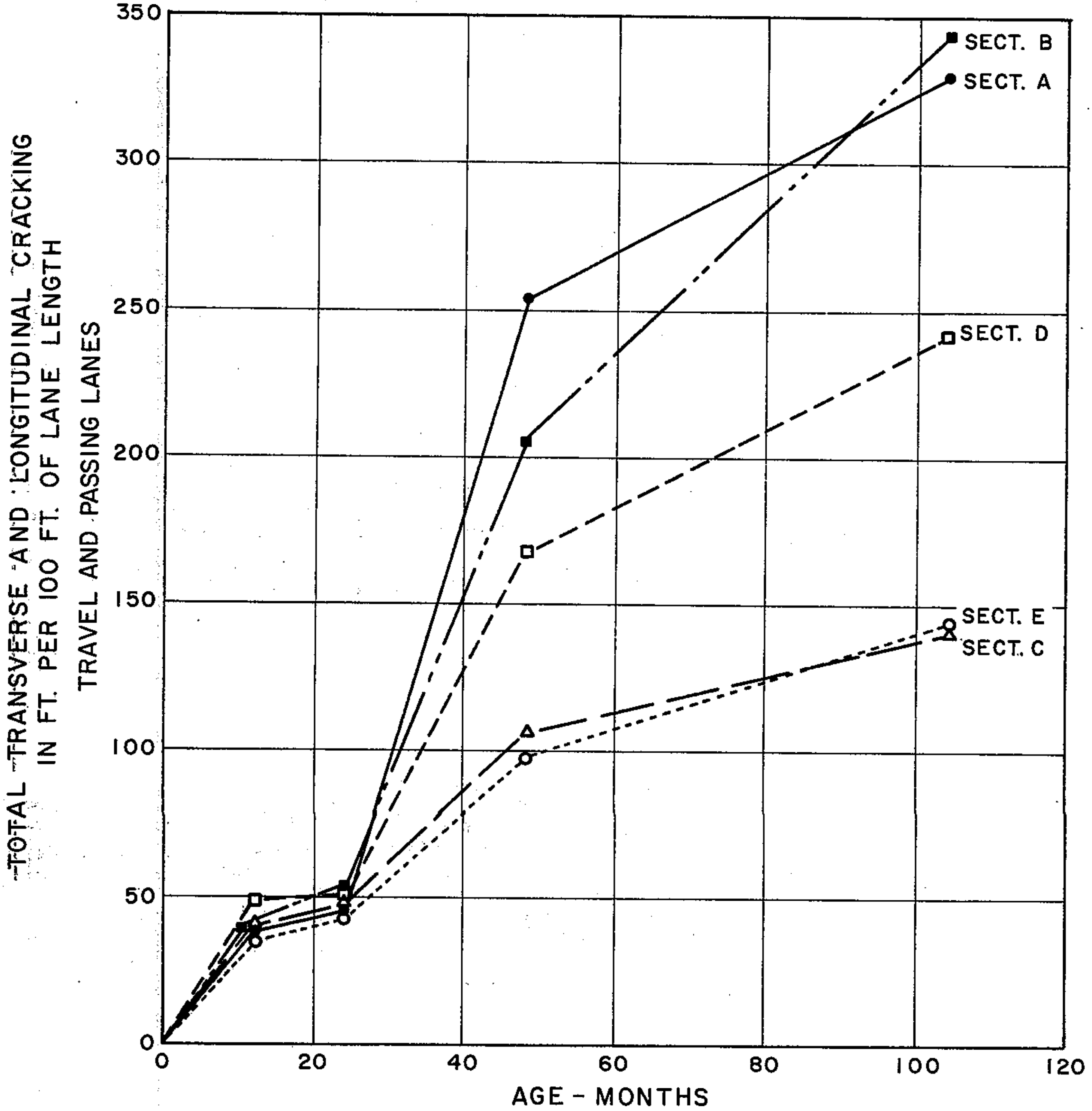


Figure 3

INCREASE IN TRANSVERSE CRACKING DURING SERVICE LIFE

CONT. 57-3TC27 ROAD 03-Pla-17, 37-B, AUB.

NEWCASTLE JOB

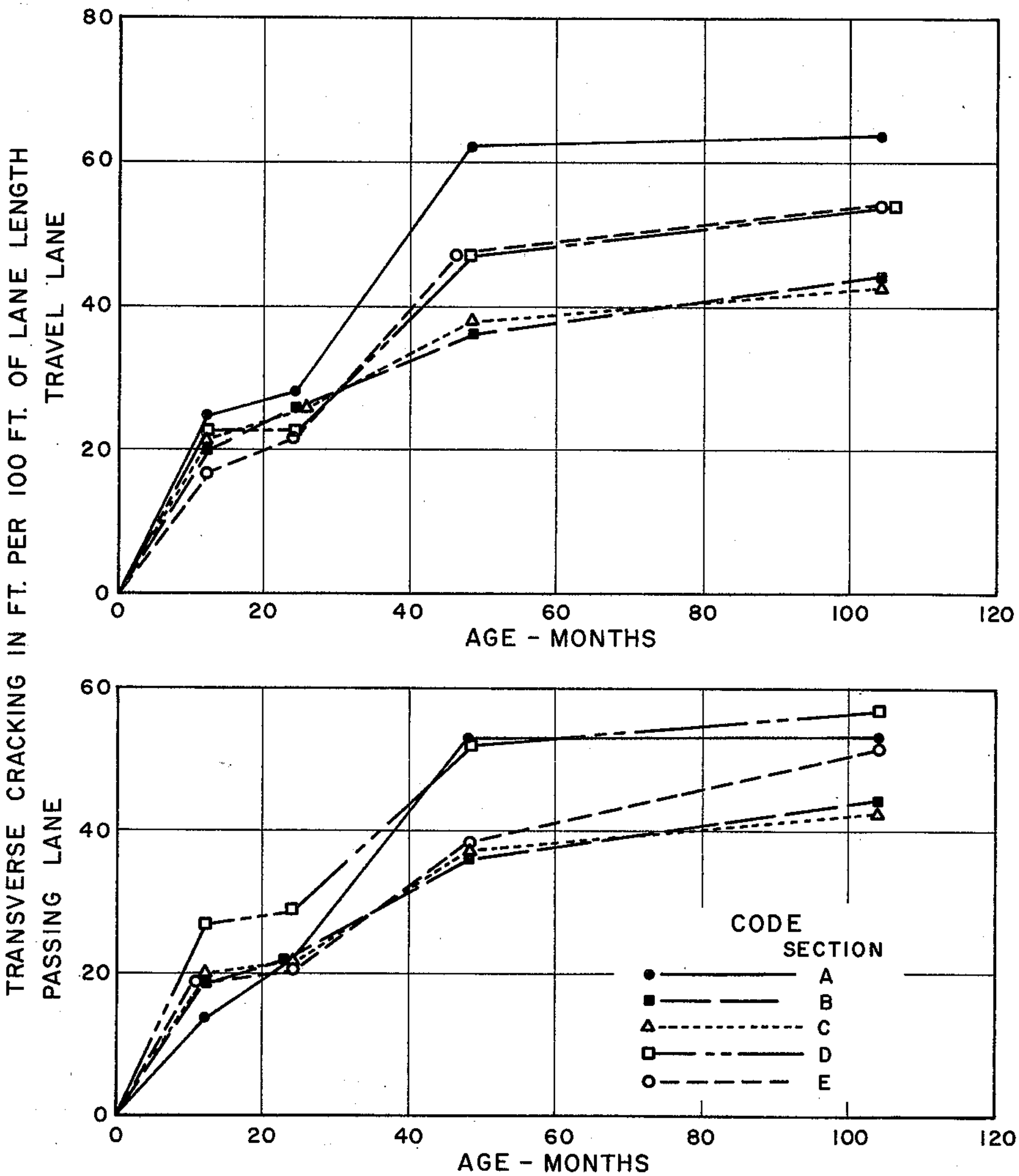
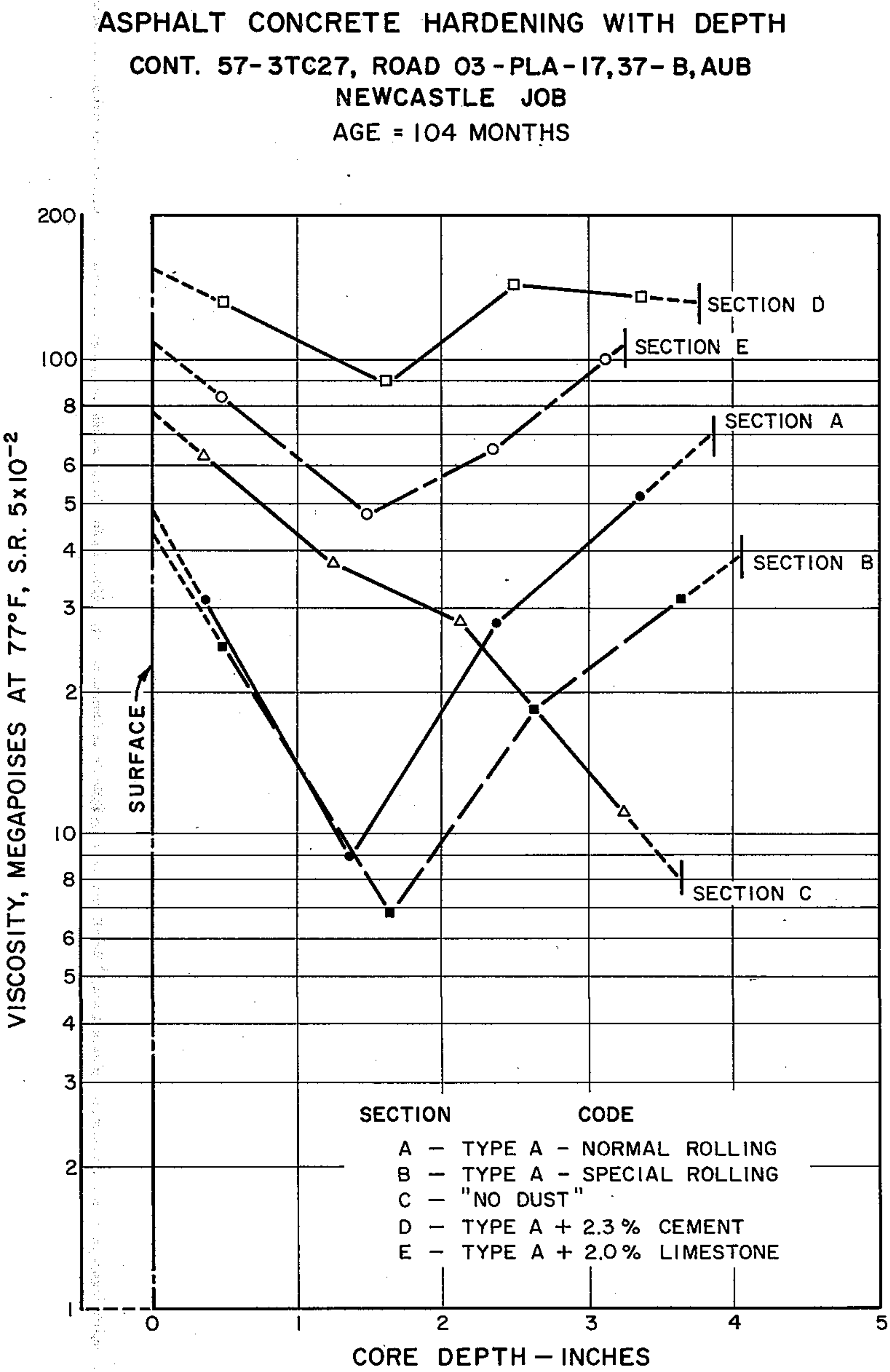


Figure 4



ASPHALT CONCRETE HARDENING WITH DEPTH
CONT. 57-3TC27, ROAD 03-PLA-17,37-B,AUB
NEWCASTLE JOB
AGE = 104 MONTHS
200
100
8
6
5
4
3
2
10
8
6
5
4
3
2
1
0
1
2
3
4
5
VISCOSITY, MEGAPOISES AT 77°F, S.R. 5x10^{-2}
CORE DEPTH — INCHES
SURFACE
SECTION D
SECTION E
SECTION A
SECTION B
SECTION C
SECTION
CODE
A — TYPE A - NORMAL ROLLING
B — TYPE A - SPECIAL ROLLING
C — "NO DUST"
D — TYPE A + 2.3 % CEMENT
E — TYPE A + 2.0 % LIMESTONE

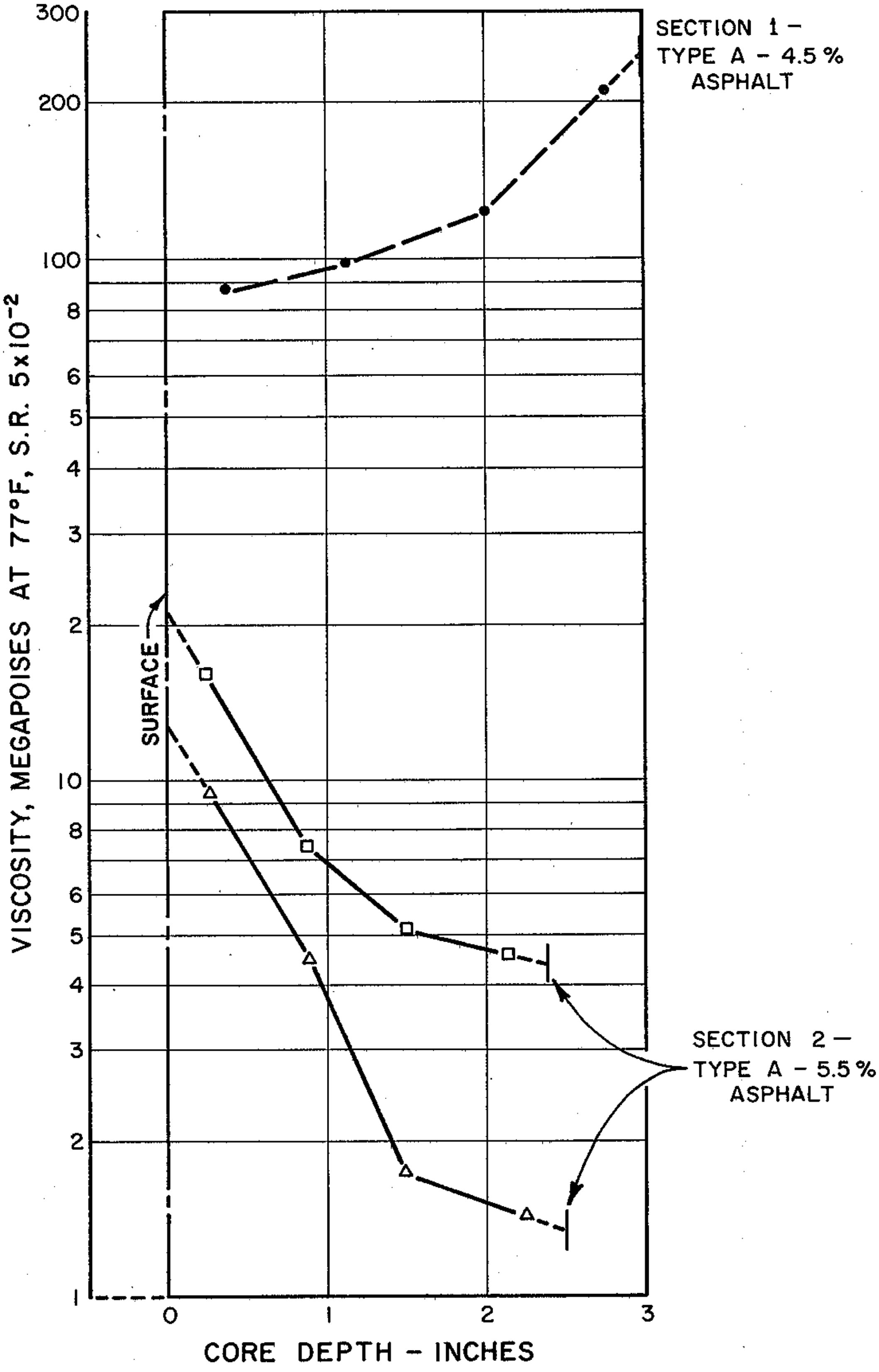
Figure 5

ASPHALT CONCRETE HARDENING WITH DEPTH

CONT. 57-3TC21, ROAD 03 - PLA - 37 - B

COLFAX JOB

AGE = 108 MONTHS



APPENDIX

Test Results for Cores Removed in January 1967 After 104 Months of Service Life From the Newcastle Job and 108 Months From the Colfax Job.

TABLE A

Physical Properties of Recovered Asphalt
Cont. 57-3TC27
Road III-Pla-17,37-B, Aub. (Newcastle Job)
Pavement Age = 104 Months

Core No.	Station and Mix Type	Air Perm.	% Asph.	Wax Grav.	% Voids	Test Results on Recovered Asphalt: Pen. 77°F	SP °F	Duct. 77°F	Viscosity 77°F .05Sec^{-1}	Viscosity M.P. .001Sec^{-1}	Shear Index	Micro Duct. mm
32472	Sta.220+00 WB-Tr.OWT Stand. Type A Normal Rolling	0	5.4	2.39	1.4	32	136	100+	14	27	0.17	18
32474	Sta.226+00 WB-Tr.OWT Stand. Type A Normal Rolling	0	4.7	2.40	0.8	26	142	79	25	60	0.23	9
Average		0	5.1	2.40	1.1	29	139	79-100+	20	44	0.20	14
32481	Sta.242+50 WB-Tr.OWT Stand. Type A Spec. Rolling	0	4.9	2.38	1.7	38	134	100+	10	18	0.15	38
32482	Sta.247+00 WB-Tr.OWT Stand. Type A Spec. Rolling	0	4.8	2.39	1.2	22	143	22	24	56	0.22	9
Average		0	4.9	2.39	1.5	30	139	22-100+	17	37	0.19	24
32478	Sta.235+50 WB-Tr.OWT "No Dust" Section	0	5.3	2.37	1.7	25	147	64	23	55	0.23	8
32480	Sta.238+00 WB-Tr.OWT "No Dust" Section	0	5.3	2.37	1.7	23	149	27	25	83	0.30	7
Average		0	5.3	2.37	1.7	24	148	45	24	69	0.27	8

TABLE A - Continued

Core No.	Station and Mix Type	Air Perm.	% Asph.	Wax Grav.	% Voids	Test Results on Recovered Asphalt						
						Pen. 77°F	SP °F	Duct. 77°F	Viscosity 77°F .05Sec^{-1}	Viscosity M.P. .001Sec^{-1}	Shear Index	Micro Duct. mm
32475	Sta.228+00 WB-Tr.OWT Stand. Type A +2.3% Cement	0	4.2	2.32	5.3	16	160	6	64	275	0.37	1
32477	Sta.233+00 WB-Tr.OWT Stand. Type A +2.3% Cement	0	4.2	2.33	4.9	16	161	6	87	355	0.36	0
Average		0	4.2	2.33	5.1	16	161	6	76	315	0.37	1
32483	Sta.243+00 EB-Tr.OWT Stand. Type A +2.0% Limestone	0	4.0	2.34	4.5	22	145	35	35	110	0.29	6
32485	Sta.248+00 EB-Tr.OWT Stand. Type A +2.0% Limestone	0	4.3	2.36	3.6	15	149	15	51	147	0.28	5
Average		0	4.2	2.35	4.1	19	147	25	43	128	0.29	6

TABLE B

Physical Properties of Cores
Cont. 57-3TC27
Road III-Pla-17,37-B, Aub.
Pavement Age = 104 Months

Core No.	Station and Mix Type	Field Asph. %	% Asph. Ext.	Grading 3/4	1/2	3/8	4	8	16	30	50	100	200
32472	Sta.220+00 WB-Tr.OWT Stand. Type A Normal Rolling	5.2	5.4	100	87	73	51	40	33	26	14	8	5
32474	Sta.226+00 WB-Tr.OWT Stand. Type A Normal Rolling	5.2	4.7	100	82	65	45	34	28	22	12	7	4
Average		5.2	5.1	100	85	69	48	37	31	24	13	8	5
32481	Sta.242+50 WB-Tr.OWT Stand. Type A Spec. Rolling	5.2	4.9	99	84	67	45	35	30	24	13	7	5
32482	Sta.247+00 WB-Tr.OWT Stand. Type A Spec.Rolling	5.2	4.8	100	83	70	47	36	29	23	12	6	4
Average		5.2	4.9	100	84	69	46	36	30	24	13	7	5
32478	Sta.235+50 WB-Tr.OWT "No Dust" Section	5.7	5.3	100	80	63	49	38	32	24	11	6	4
32480	Sta.238+00 WB-Tr.OWT "No Dust" Section	5.7	5.3	99	79	67	48	38	31	23	11	5	4
Average		5.7	5.3	100	80	65	49	38	32	24	11	6	4

TABLE B - Continued

Core No.	Station and Mix Type	Field Asph. %	% Asph. Ext.	Grading									
				3/4	1/2	3/8	4	8	16	30	50	100	200
32475	Sta.228+00 WB-Tr.OWT Stand. Type A +2.3% Cement	4.5	4.2	100	84	67	48	37	31	24	11	6	4
32477	Sta.233+00 WB-Tr.OWT Stand. Type A +2.3% Cement	4.5	4.2	95	79	65	48	38	31	24	12	6	4
Average		4.5	4.2	98	82	66	48	38	31	24	12	6	4
32483	Sta.243+00 EB-Tr.OWT Stand. Type A +2.0% Limestone	4.5	4.0	100	81	67	46	37	30	23	12	7	5
32485	Sta.248+00 EB-Tr.OWT Stand. Type A +2.0% Limestone	4.5	4.3	100	80	67	48	38	31	24	12	7	5
Average		4.5	4.2	100	81	67	47	38	31	24	12	7	5

TABLE C

Physical Properties of Recovered Asphalt
Cont. 57-3TC21
Road III-Pla-37-B (Colfax Job)
Pavement Age = 108 Months

Core No.	Station and Mix Type	Air Perm.	% Asph.	Wax Grav.	% Voids	Test Results on Recovered Asphalt						
						Pen. 77°F	S.P. °F	Duct. 77°F	Viscosity 77 F .05Sec⁻¹	Viscosity M.P. .001Sec⁻¹	Shear Index	Micro Duct. mm
32486	Sta.306+00 EB-Tr.OWT Standard Type A	73	4.3	2.32	5.3	13	147	90	92	158	0.14	0
32487	Sta.310+00 EB-Tr.OWT Standard Type A	0	4.2	2.31	5.4	13	137	17	97	216	0.20	2
Aver.		37	4.3	2.32	5.3	13	142	54	95	187	0.17	1
32488	Sta.313+50 EB-Tr.OWT Standard Type A	0	5.4	2.42	0.0	50	128	100+	4.5	4.5	0.00	56
32489	Sta.317+00 EB-Tr.OWT Standard Type A	0	5.2	2.41	0.4	34	137	100+	11.9	11.9	0.00	116
32490	Sta.320+50 EB-Tr.OWT Standard Type A	0	5.4	2.40	0.8	50	132	100+	5.7	5.7	0.00	105
Aver.		0	5.3	2.41	0.4	45	133	100+	7.4	7.4	0.00	92

TABLE D

Physical Properties of Cores
Cont. 57-3TC21
Road III-Pla-37-B (Colfax Job)
Pavement Age = 108 Months

Core No.	Station and Mix Type	Field Asph. %	% Asph. Ext.	Grading									
				3/4	1/2	3/8	4	8	16	30	50	100	200
32486	Sta.306+00 EB-Tr.OWT Standard Type A	4.5	4.3	100	85	68	46	34	27	19	11	7	5
32487	Sta.310+00 EB-Tr.OWT Standard Type A	4.5	4.2	100	83	67	46	35	27	19	11	7	5
Average		4.5	4.3	100	84	68	46	35	27	19	11	7	5
32488	Sta.313+50 EB-Tr.OWT Standard Type A	5.5	5.4	100	85	66	44	33	26	18	10	7	5
32489	Sta.317+00 EB-Tr.OWT Standard Type A	5.5	5.2	100	83	64	42	33	26	18	10	7	5
32490	Sta.320+50 EB-Tr.OWT Standard Type A	5.5	5.4	100	83	64	45	34	26	18	10	7	5
Average		5.5	5.3	100	84	65	44	33	26	18	10	7	5

HIGHWAY RESEARCH REPORT

REDUCTION OF ACCIDENTS BY PAVEMENT GROOVING

John L. Beaton
Ernest Zube and John Skog

Presented at the Western Summer Meeting
of the Highway Research Board
Denver, Colorado
August, 1968

68-24

STATE OF CALIFORNIA

TRANSPORTATION AGENCY

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

MATERIALS AND RESEARCH DEPARTMENT

RESEARCH REPORT

NO. M & R 633126

Prepared in Cooperation with the U.S. Department of Transportation, Bureau of Public Roads August, 1968

STATE OF CALIFORNIA
TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

REDUCTION OF ACCIDENTS BY PAVEMENT GROOVING

By

J. L. Beaton
Materials and Research Engineer

E. Zube
Assistant Materials and Research Engineer

John Skog
Senior Materials and Research Engineer

Presented at the Western
Summer Meeting
of the Highway Research Board
Denver, Colorado
August 1968

[illegible]

REFERENCE: Beaton, J. L., Zube, E., and Skog, J. B. "Reduction of Accidents by Pavement Grooving", State of California, Department of Public Works, Division of Highways, Materials and Research Department. Research Report 633126-2, August 1968.

ABSTRACT: Providing and maintaining a skid resistant surface on concrete pavements is discussed. Studies of the effect of grooving the pavement to reduce wet weather accidents were conducted. The objective of the studies was to determine the efficiency of serrations in raising the skid resistance, to determine resistance of a grooved pavement to wear and polish of traffic, and to determine the extent of reduction in wet weather accidents by serration of the pavement. Results show that pavement grooving parallel to the centerline will reduce the wet weather accident rate in low friction value areas of PCC pavements. Friction value is raised following grooving. Wear and polish of grooved areas appear to depend on characteristics of the pavement.

KEY WORDS: Pavements, pavement skidding characteristics, pavement surfaces, grooving, accidents, accident rates, wear, polishing.

ACKNOWLEDGMENT

This paper is based on data collected during a research project financed by the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads. The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

INTRODUCTION

Providing and maintaining a skid resistant surface is of primary importance to the proper performance of any highway. All types of pavement surface will eventually show some reduction in coefficient of friction values during their service life. This reduction is caused by wear and polish of traffic, especially by heavy trucks.

Several years ago California Division of Highways accident analysis showed that some sections of concrete freeways, especially on curves, were having an unusual number of accidents occurring during wet or rainy weather. After considering the use of acid treatment of the surface or the application of a coal tar-epoxy screening seal coat or some other thin organic overlay, it was decided to study the effect of grooving the pavement. The objectives of the program are as follows:

1. To determine the efficiency of serration in raising the skid resistance.
2. To determine the resistance of a grooved pavement to wear and polish of traffic.
3. To determine the extent of reduction in wet weather accidents in critical areas by serration of the pavement.

MEASUREMENT OF SKID RESISTANCE

The California Skid Tester used in determining the coefficient of friction of pavement surfaces has been previously described[1]. The presently used test method is shown in the attached appendix. The tester has been calibrated with the towed trailer equipment constructed by Professor R. A. Moyer of the University of California, Institute of Transportation[2]. Previous studies by Professor Moyer and others indicated that the skid resistance value for any given surface approaches a low figure when the brakes are locked on a vehicle having smooth tread tires and traveling at speeds of fifty miles per hour on a wet pavement. Therefore, in the correlation program, the coefficient of friction values obtained from Moyer's unit using locked wheels, smooth tires, wet pavement and a speed of fifty miles per hour were compared to our readings obtained under identical operating conditions.

We are presently using a value of f=0.25 as the minimum requirement for indicating the need for remedial action, and a minimum of f=0.30 for new PCC pavement. An active program is underway to study the adequacy of these values, especially the figure for remedial action. The program involves the use of recommended minimum friction values from other sources, and an accident frequency correlation with skid resistance of the pavement surface.

C. G. Giles[3] on the basis of a comprehensive accident analysis in England has provided a set of suggested values of skid resistance for use with the British Portable

Tester. A comprehensive correlation program was performed by us in order to obtain the relation between the California Tester and the British Portable Tester.

On the basis of the correlation, a comparison of the recommended British values with the tentative California minimum figure is shown in Figure 1. Also shown is the Virginia minimum figure which was attained by using the correlation chart of D. C. Mahone[4] which provides an approximate correlation between the British Portable Tester and the Virginia test car at 40 mph.

Preliminary studies in California on a wet weather accident frequency correlation with skid resistance indicates that most single car accidents occurred on curves, the average value for the friction factor being f=0.22. However, twenty-eight percent of the accidents that occurred on curves were on pavements of f=0.25-0.28 range. The maximum value attained in this study was f=0.28.

On the basis of these results, it is concluded that the present f=0.25 remedial action value is a minimum figure, and it appears that the value may be too low for curves of rather short radius. A better value may be f=0.28 which is the same as the British minimum for all sites. Further studies are underway.

PATTERN STUDIES

Grooves may be cut in the pavement in either a longitudinal (parallel to the centerline), transverse or skewed direction. All grooving (except for a few short experimental sections) to date on State Highways has been performed in a longitudinal direction. We are of the opinion that this leads to increased lateral stability, and tends to guide the vehicle through a critical curve area. This has been confirmed by studies performed in Texas[5]. However, studies in England[6] indicate that grooving perpendicular to the centerline is better in this connection - further effort will be required to resolve the problem.

Groove patterns vary. The most common type is rectangular in form and may be varied in width and depth and distance between centers of grooves. Other types have rectangular form, but the bottom is partially rounded, and the edges at the pavement surface are also rounded. Others have a large V cut separated by smaller V cuts. Figure 2 shows two types of patterns.

A number of patterns have been used in our serration work to date. This was done in order to determine the increase in the friction factor, wear resistance, and possible vehicle handling problem. In all cases the grooves are all in a longitudinal direction. Figure 3 shows the patterns used on the various projects, and Table A the increase in the friction value after grooving and the change during service life. Figure 4 shows the effect of grooving on the average coefficient of friction value for the various PCC pavement projects.

In all cases the friction value is raised by pavement grooving. However, it appears that the nature of the existing concrete surface and the type of pattern effect the degree of improvement in the friction value. As an example there is a much greater improvement in the friction value for project H than on projects F and G for a 1/8" x 1/8" on 1" centers with a rectangular groove. This is also confirmed by the results from projects J and K where two different patterns are compared on two different projects.

The type of pattern on any specific project effects the degree of improvement. On project K three different Christensen patterns (Table A) were placed in consecutive one hundred foot test sections in the travel lane. The original coefficient of friction values were identical, but two of the patterns produced a very high degree of improvement as compared to the third pattern.

Project I in District 07 is an aged asphalt concrete pavement. The surface was rather dry in appearance and quite brittle. Therefore, it was decided to groove this pavement using 1/4" x 1/4" grooves on 1" centers. Shortly after completion several complaints were received from drivers of motorcycles and light cars. The complaints were that the vehicle tended to "track" and appeared to be caught in a manner resembling being caught in streetcar tracks. This was confirmed by Highway Patrolmen. On the other hand, Christensen Style 15 with V cuts 1/4" wide on the Placerita Canyon Bridge provided no problems with test motorcycles driven up to 70 mph. There was some vibration up to 50 mph with Style 9, but this tended to fade out at higher speeds. Style 9, see Table A project J, has 3/16" wide rectangular grooves with rounded bottom and edges. These studies indicate that rectangular longitudinal grooves should not be wider than 1/8" in order to prevent possible problems from motorcycles and light passenger cars. However, V cuts do not appear to cause problems although 1/4" wide at the surface.

A very important characteristic of any treatment for raising the existing friction value is its resistance to wear and polish of traffic. Results of variations of friction measurements with time on various grooved projects are shown in Table A and Figure 5. Not sufficient time has elapsed on the majority of the projects to draw any firm conclusions. It appears, however, that the nature of aggregate and mortar strength may influence the resistance to wear and polish of the grooved areas. However, it is interesting to note that projects A and B cover the travel lanes of heavily travelled freeways having a high percentage of trucks. All the projects shown in Figure 5 are in snow free areas. Project H in Table A is in a partial snow region where chains may be required. After the first winter the surface does not appear to be damaged by chain action. This project will be closely watched since project M in Table B has shown considerable spalling between the grooves which are on one inch centers. This spalling has been caused by chain action and has resulted in some complaints in regards to controllability of a car even under dry pavement conditions.

ACCIDENT STUDIES

A summary of all of the presently available accident data are shown in Tables B, C and D. Six of these locations were on urban freeways in the vicinity of Los Angeles. Accident data was also reviewed for comparison purposes on a mile of unserrated asphalt concrete freeway, see Table D. The Los Angeles projects had one year before and after accident analysis periods. An additional project M on Interstate 80 near the Nevada state line had a two year period for before and after accident analysis. This freeway is rural and required longer periods to obtain meaningful data. In the case of the Los Angeles area freeways, the number of wet or rainy days was determined in both the before and after accident periods. There were 30 wet days in the before period and approximately 15 wet days in the after period. Fifteen additional wet days were accumulated from the following year and the accidents on these days were added to the after period.

A study of Table B indicates that the total accidents were reduced 78 percent; of this, wet pavement accidents were almost completely eliminated (96 percent) and dry pavement accidents dropped 32 percent.

The reduction in dry weather accidents, if confirmed by further observation, appears to be significant. There is no reason to doubt that the dry friction value of these pavements was sufficiently high. In our opinion the decrease in dry weather accidents may be the result of the ability of the grooves to "track" or aid as a guide for a vehicle nearing an out of control condition in the curve area. Such loss of control would most commonly be caused by entering the curve at excessive speed and then rapid deceleration within the curve area. Such action could cause loss of control. The longitudinal grooves by acting as "tracks" could resist lateral movements and add stability to the vehicle. In the case of the wet pavement condition we may, therefore, assume that longitudinal grooving in curve areas not only increases the friction factor, but also acts as a stabilizer against lateral instability. It probably also serves as a quick surface drain to minimize any water buildup on the pavement.

Table C shows the exposure in million vehicle miles, accident rates, and other information. Both wet and dry pavement accident rates were calculated relative to the number of wet or dry days. These rates could not be calculated at the Interstate 80 location, project M, since the number of wet days was not available.

All of the accident rates on wet days were much higher than the average state highway rates at both urban and rural locations. Since the number of wet days is relatively minor in southern California, the resulting exposure is small. When this is divided into the overall large number of accidents occurring on wet pavement, the result is an unusually high rate. All locations (excepting one) had higher than average total accident rates in the before grooving period. The concrete surfaced urban freeways all had below average (≪1.61) rates in the after period. The two rural locations (both concrete surfaced) still had higher than average total accident rates (≪1.00) despite sizable drops in rates after pavement serration.

For comparison purposes the accident rate on a one mile stretch of asphaltic concrete pavement just south of the serrated project N was compared with the unserrated control section. The results are shown in Table D and clearly indicate the excellent reduction in wet weather accidents following grooving. In the same period the control section had a gain in wet weather accidents.

It is proposed to continue this accident analysis, and periodical skid resistance surveys to determine possible increase in accidents as the friction values change during service life.

COST OF GROOVING

On seven jobs in District 07 the cost of grooving was in the range of seven to nine cents per square foot. In some other Districts the cost is somewhat higher. The best estimate is approximately ten cents per square foot.

SUMMARY

In summary it appears that pavement grooving performed in a direction parallel to the centerline will definitely reduce the wet weather accident rate in low friction value areas of PCC pavements. Excellent reduction of wet weather accidents occurred after grooving of an old asphalt concrete pavement. However, this pavement was very hard and brittle, and we do not recommend grooving of normal asphalt concrete pavement, since kneading by traffic may rapidly close the grooves. It seems preferable to apply a screening seal coat, slurry seal coat or dense or open graded blanket.

The friction value is raised following grooving. The rate of change in friction value by wear and polish of the grooved area appears to depend on the characteristics of the original concrete pavement, since two pavements with heavy truck traffic showed little change in friction values after a number of years of service. On the other hand some pavements show quite rapid drops after only seventeen months of traffic. Further tests are required.

Motorcycle and light car tests clearly indicate that 1/4" x 1/4" grooves will create problems in vehicle control. It is recommended that cuts no greater than 1/8" x 1/8" be used if vertical grooves are cut in the pavement. However, 1/8" deep x 1/4" wide V grooves do not appear to create any problems. Further studies are required before any specific spacing may be recommended. However, since approximately equal accident reductions were noted for 1/2" and 3/4" spacing, it is recommended that 1/8" x 1/8" on 3/4" centers be used. It is highly desirable that further areas be grooved with a series of patterns as was done on the Ventura project in order to determine effectiveness in raising the original coefficient of friction, and resistance to wear and polish under equivalent concrete and traffic conditions.

REFERENCES

1. Hveem, F. N., Zube, E. and Skog, J.
"California Type Skid Resistance Tester for Field and Laboratory Use"
Proceedings First International Skid Prevention Conference, Part II, 1959, p. 365.

2. Moyer, R. A.
"Recent Development in the Measurement of Road Roughness and Skid Resistance"
Proceedings Association of Asphalt Paving Technologists, Vol. 20, 1951, p. 42.

3. Giles, C. G., Sabey, B. E. and Cardew, K. H. F.
"Development and Performance of the Portable Skid Resistance Tester"
British Road Research Laboratory Technical Paper No. 66, 1964.

4. Mahone, D. C.
"A Correlation of the British Portable Tester and the Virginia Skid Test Car"
Virginia Council of Highway Investigation and Research, May 1961.

REFERENCES (con't.)

5. McCullough, F., and Hilgers, H. F.
"Slick When Wet and Field Evaluation of Saw Cut Method"
Texas Highways, Vol. 10, No. 1, p. 9, 1963.

6. Blake, L. S.
"Recent Research and Development on Concrete Roads, 1964-1967"
The Concrete Society, Limited
Terminal House, Grosvenor Gardens, London.

TABLE A

Change in Average Friction Values Following Grooving

Project No.	Pavement Type	Location	AADT/1000	Serration Pattern	Age Mos.	Ave. Friction Value
A	PCC Bridge Deck	10-Sta-4-A	24	Rectangular Grooves 1/8"x1/8" on 3/8" Centers	Before	0.26
					After	0.33
					45	0.33
					101	0.36
B	PCC Bridge Deck	04-Ala-7-Alb	80	Rectangular Grooves 1/8"x1/8" on 3/8" Centers	Before	0.26
					After	0.32
					41	0.28
C	PCC	06-Kern-5-PM6.94-7.47	16	Rectangular Grooves 1/8"x1/8" on 3/8" Centers	Before	0.19
					After	0.32
					67	0.34
D	PCC	07-Ora-5-PM23.3-23.6	45	Rectangular Grooves 1/8"x1/8" on 1/2" Centers	Before	0.25
					After	0.35
					17	0.30
E	PCC	07-LA-5 PM29.5-30.0	104	Rectangular Grooves 1/8"x1/8" on 3/4" Centers	Before	0.23
					After	0.31
					17	0.27
F	PCC	07-LA-405 PM2.1-2.6	131	Rectangular Grooves 1/8"x1/8" on 1" Centers	Before	0.20
					After	0.24
					17	0.22

TABLE A

Change in Average Friction Values Following Grooving

Project No.	Pavement Type	Location	AADT/1000	Serration Pattern	Age Mos.	Ave. Friction Value
G	PCC	07-LA-405 PM3.8-4.1	139	Rectangular Grooves 1/8"x1/8" on 1" Centers	Before	0.19
					After	0.21
H	PCC	03-Pla,Nev-80 Var.	9	Rectangular Grooves 1/8"x1/8" on 1" Centers		
H-1		E.B. Lane PM42.56-42.77			Before	0.24
					After	0.37
					12 Mo.	0.34
H-2		W.B. Lane PM45.45-45.60	9	"	Before	0.25
					After	0.32
					12 Mo.	0.29
H-3		W.B. Lane PM5.00-5.27	9	"	Before	0.19
					After	0.29
					12 Mo.	0.27
H-4		E.B. Lane PM6.55-6.65	9	"	Before	0.15
					After	0.30
					12 Mo.	0.25
H-5		W.B. Lane PM9.01-9.19	9	"	Before	0.19
					After	0.30
					12 Mo.	0.27

TABLE A

Change in Average Friction Values Following Grooving

Project No.	Pavement Type	Location	AADT/1000	Serration Pattern	Age Mos.	Ave. Friction Value
I	AC	07-LA-101 PM8.8-9.3	134	Rectangular Grooves 1/4"x1/4" on 1" Centers	Before	0.23
					After 17	0.28 0.29
J	PCC	07-LA-14-27.89 Placerita Canyon Bridge	13	Christensen Co. Style #9	Before	0.16
					After	0.26
				Christensen Co. Style #15	Before	0.16
					After	0.33
K	PCC	07-Ven-101	21	Christensen Co. Style #6	Before	0.20
					After	0.37
				Christensen Co. Style #9	Before	0.20
					After	0.31
				Christensen Co. Style #15	Before	0.19
					After	0.37

TABLE B

Effect on Number of Accidents Following Grooving

Proj. No.	Location & Pvt. Type	Serration Pattern	Curvature Radius Ft.	Curvature Dir.	AADT/1000	Accidents Before Wet	Before Dry	Before Tot.	After Wet	After Dry	After Tot	% Change Wet	% Change Dry	% Change Tot.
D	07-Ora-5 PM23.3-23.6 PCC	1/8"x1/8"on1/2" Centers-Rect. Grooves	2000'	Rt.	45	46	4	50	1	7	8	-98	+75	-84
E	07-LA-5 PM29.5-30.0 PCC	1/8"x1/8"on3/4" Centers-Rect. Grooves	2000'	Lt.	104	12	6	18	2	2	4	-83	-67	-78
L	07-LA-10 PM22.6-22.8 PCC	1/8"x1/8"on3/4" Centers-Rect. Grooves	1020'	Lt.	164	26	16	42	0	6	6	-100	-63	-86
F	07-LA-405 PM2.1-2.6 PCC	1/8"x1/8"on 1" Centers-Rect. Grooves	Tangent		131	21	9	30	0	11	11	-100	+22	-63
G	07-LA-405 PM3.8-4.1 PCC	1/8"x1/8"on 1" Centers-Rect. Grooves	3000'	Rt.	139	4	6	10	0	4	4	-100	-33	-60
M	03-Nev-80(1) PM19.8-20.2 PCC	1/8"x1/8"on 1" Centers-Rect. Grooves	1400'	Lt.	9	5	9	14	0	6	6	-100	-33	-57
N	07-LA-101 PM8.8-9.3 AC	1/4"x1/4"on 1" Centers-Rect. Grooves	2050' 2052'	Revers-ing	134	139	55	194	6	35	41	-96	-36	-79
Total						253	105	358	9	71	80	-96	-32	-78

(1) Two year before and after period. All others one year.

TABLE C

Effect on Accident Rate Following Grooving

Proj. No.	Pvt. Type	U-Urban R-Rural	State Ave. Acc. Rate	Before						After					
				Wet		Dry		Total		Wet		Dry		Total	
				MVM	Rate	MVM	Rate	MVM	Rate	MVM	Rate	MVM	Rate	MVM	Rate
D	PCC	R	1.00	0.18	255.56	1.96	2.04	2.14	23.36	0.20	5.00	2.26	3.10	2.46	3.25
E	PCC	U	1.61	0.77	15.58	8.54	0.70	9.31	1.93	0.78	2.56	8.71	0.23	9.49	0.42
L	PCC	U	1.61	0.49	53.06	5.46	2.93	5.95	7.06	0.49	0.00	5.50	1.09	5.99	1.00
F	PCC	U	1.61	0.97	21.65	10.80	0.83	11.77	2.55	0.98	0.00	10.97	1.00	11.95	0.92
G	PCC	U	1.61	0.59	6.78	6.64	0.90	7.23	1.38	0.63	0.00	6.98	0.57	7.61	0.53
M	PCC	R	1.00	-	-	-	-	1.02	13.73	-	-	-	-	1.31	4.58
I	AC	U	1.61	2.04	68.14	22.78	2.41	24.82	7.82	2.01	2.99	22.45	1.56	24.46	1.68
Total for PCC Pvts.			1.48	3.00	36.33	33.40	1.23	37.42	4.38	3.08	0.97	34.42	0.87	38.81	1.00

Note

MVM = Million Vehicle Miles

$$\text{Rate} = \frac{\text{Number of Accidents}}{\text{MVM}}$$

TABLE D

Comparison of Number of Accidents on Grooved and Control Asphalt Concrete Pavement

Proj. No.	Location & Pvt. Type	Serration Pattern	Curvature		AADT	Accidents								
						Before			After			% Change		
			Radius Ft.	Dir.	1000	Wet	Dry	Tot.	Wet	Dry	Tot.	Wet	Dry	Tot.
I-1	07-LA-101 PM7.8-8.8 AC	No Serration (Control)	Var.	Revers-ing	123	36	59	95	41	75	116	+14	+27	+22
I	07-LA-101 PM8.8-9.3 AC	1/4"x1/4"on 1" Centers-Rect. Grooves	2050 2052	Revers-ing	134	139	55	194	6	35	41	-96	-36	-79

CORRELATION STUDIES ON MINIMUM FRICTION VALUE FOR REMEDIAL ACTION

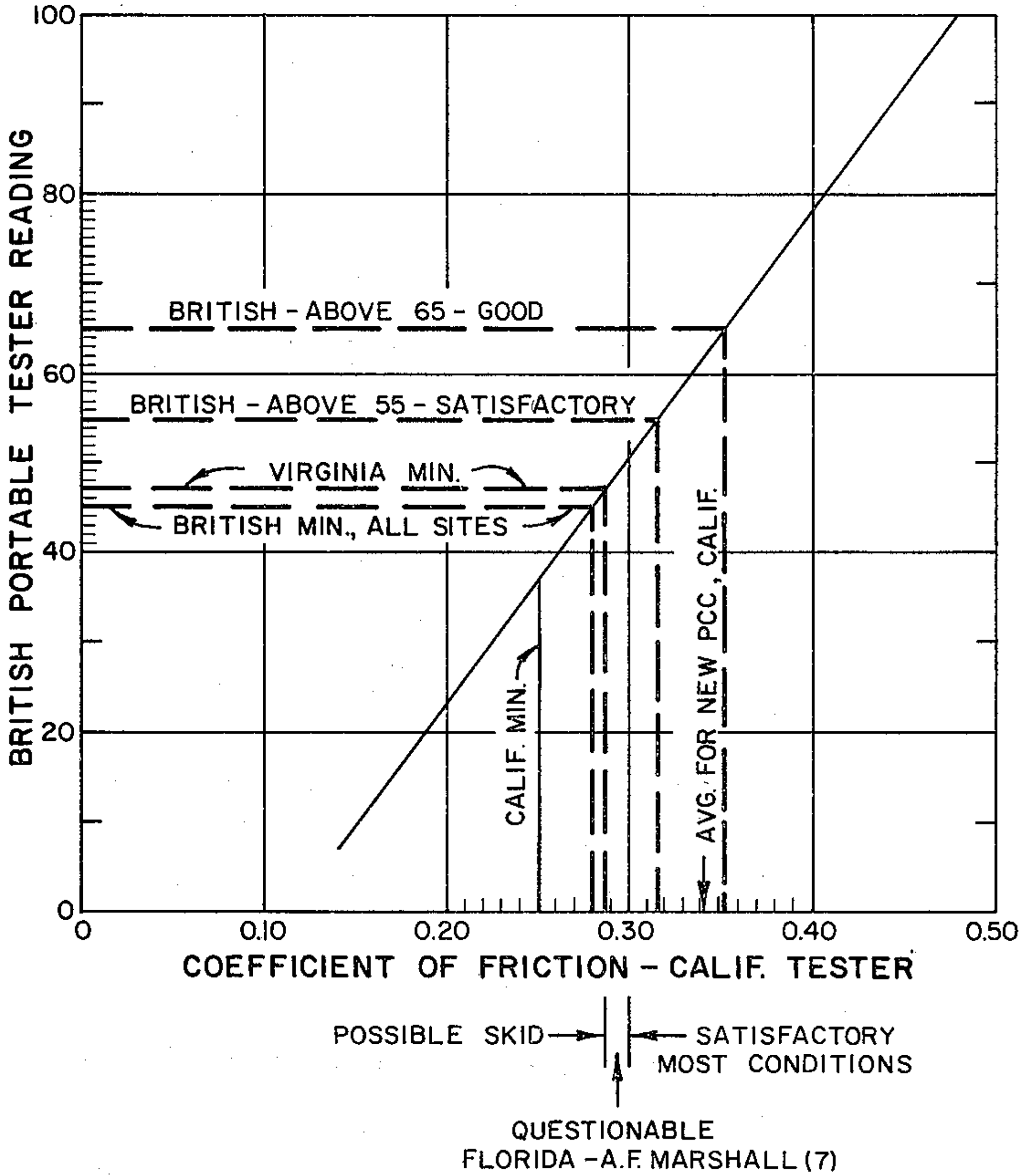
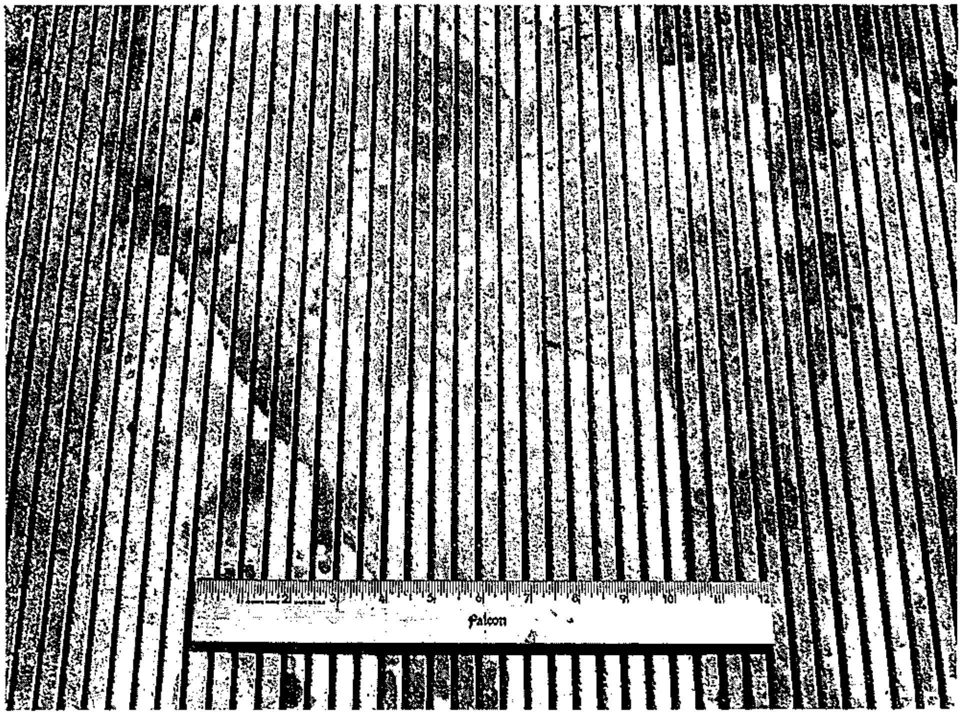
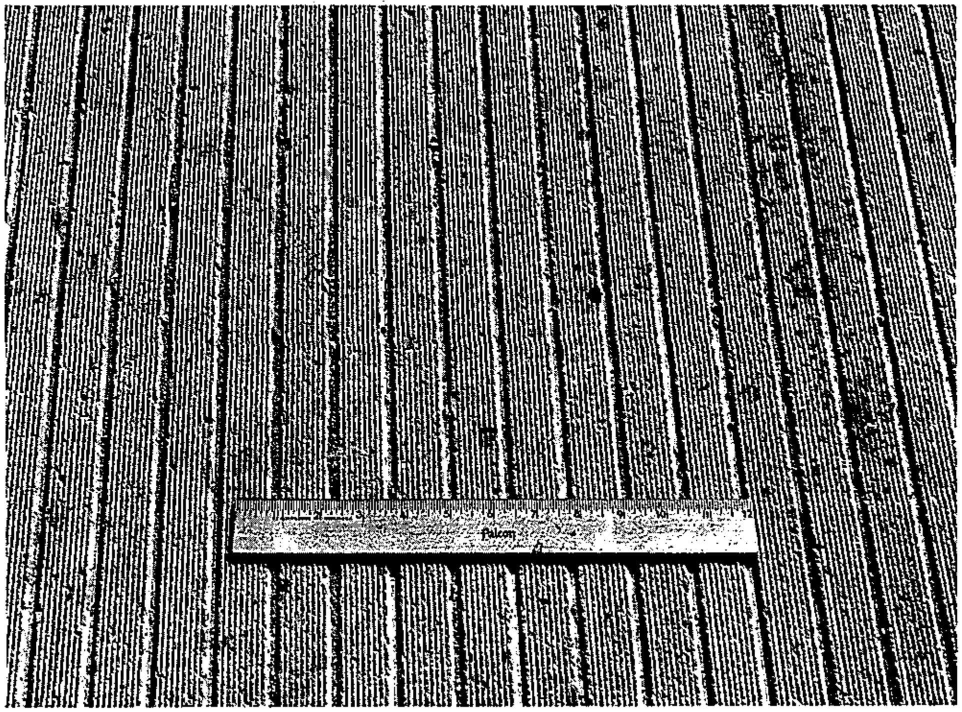


Figure 2

Photographs of Several Patterns Used on Various Projects



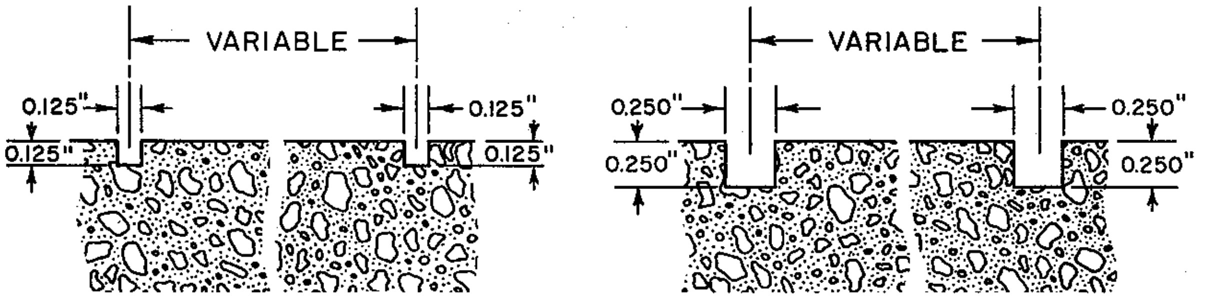
Rectangular Grooves



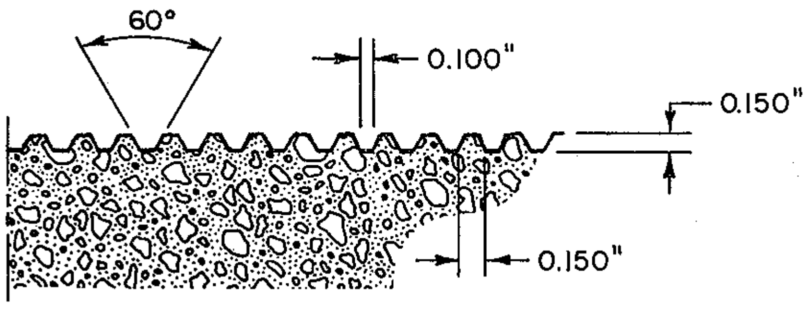
Style 15

Figure 3

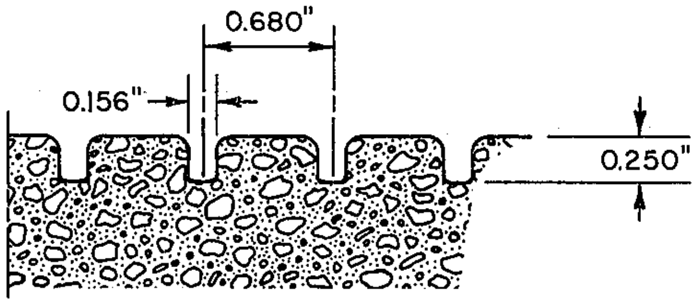
GROOVING PATTERNS USED ON VARIOUS PROJECTS



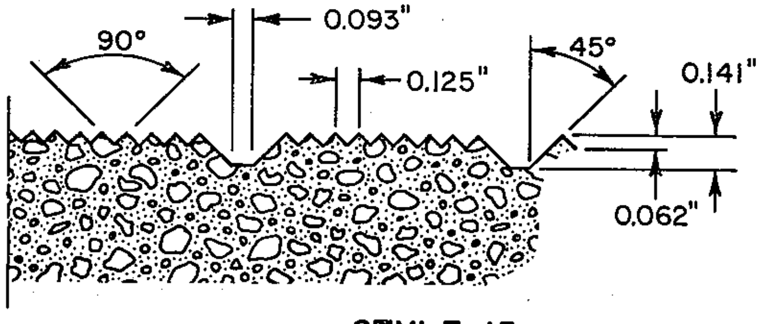
RECTANGULAR PATTERNS



STYLE 6 - 47 GROOVES PER FT.



STYLE 9



STYLE 15

Figure 4

EFFECT OF GROOVING PATTERN ON AVERAGE COEFFICIENT OF FRICTION VALUE OF PCC PAVEMENTS

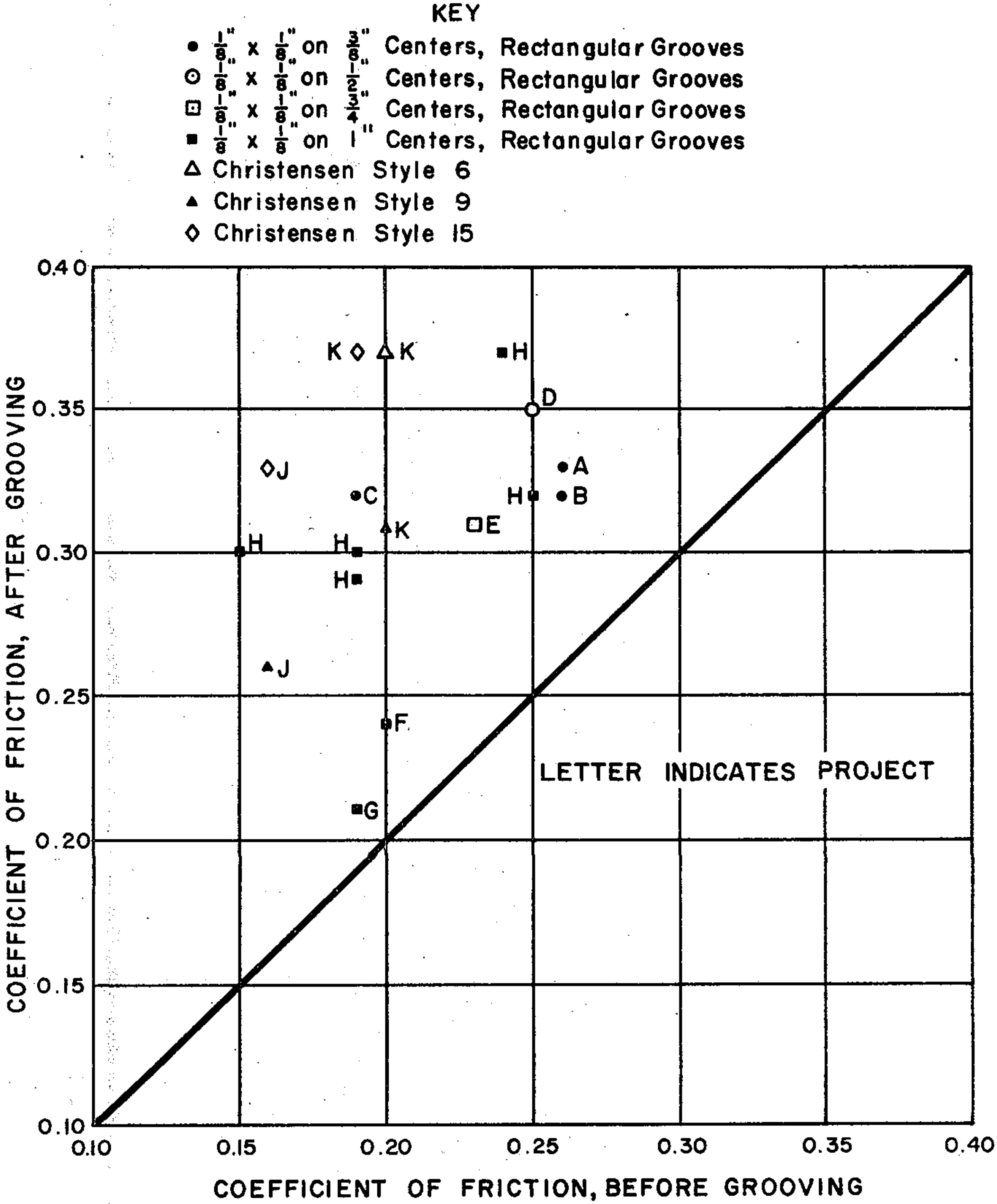
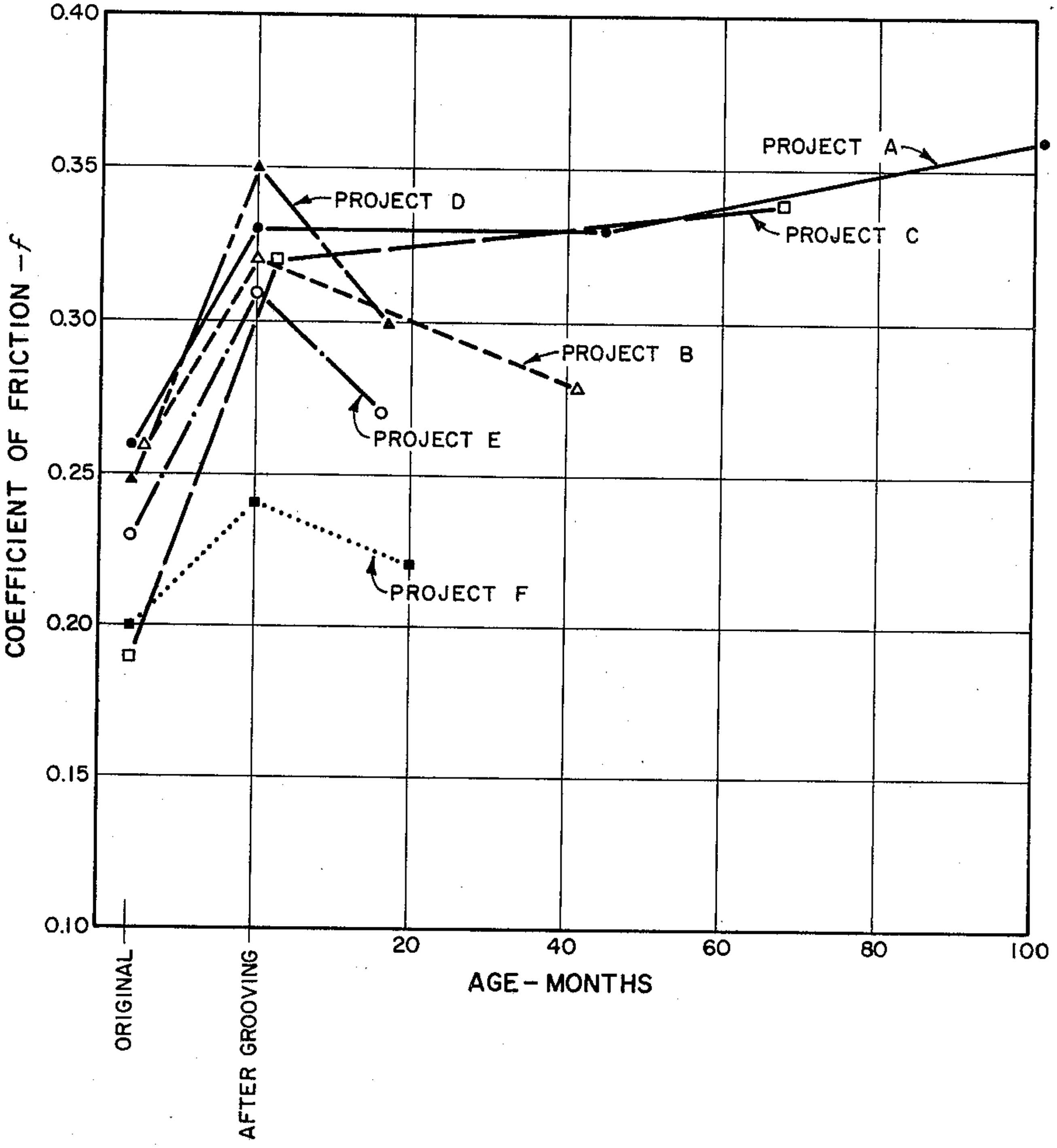


Figure 5

CHANGE IN FRICTION VALUES FOLLOWING GROOVING OF PCC PAVEMENTS



State of California
Department of Public Works
Division of Highways

MATERIALS AND RESEARCH DEPARTMENT

Test Method No. Calif. 342-C
October 3, 1966

METHOD OF TEST FOR PAVEMENT SURFACE SKID RESISTANCE

Scope

This method describes the apparatus and procedure for obtaining surface skid resistance values of Bituminous and Portland Cement Concrete pavements.

Procedure

A. Apparatus

1. Skid test unit.

a. Reference is made to Figures I through III in connection with the following description of the construction of the test unit. A 4.80/4.00 x 8, 2-ply tire with (25 ± 2 psi) air pressure (A), manufactured with a smooth surface, together with rim, axle and driving pulley is mounted on a carriage (B). The tire is brought to desired speed by motor (H). The carriage moves on two parallel guides (C), and the friction is reduced to a low uniform value by allowing three roller bearings fitted at 120° points to bear against the guide rod at each corner of the carriage. The bearing assembly may be noted on Figure III (D). The two guide rods (C) are rigidly connected to the end frame bars (E). The front end of this guide bar frame assembly is firmly fastened to a restraining anchor. The bumper hitch provides for swinging the skid tester to the right or left after positioning the vehicle. The rear end of the frame assembly is raised by a special adjustable device (F), Figure II, so as to hold the tire ¼-inch off the surface to be tested. This device is so constructed that the tire may be dropped instantaneously to the test surface by tripping the release arm (G), Figure II. Tachometer (K) indicates the speed of the tire.

2. Hitch for fastening unit to vehicle.
3. Special level to determine grade of pavement.

a. A 28″ long standard metal carpenter's level, Fig. IV, is fitted at one end with a movable gauge rod which is calibrated in % of grade.

B. Materials

1. Glycerine.
2. Water.
3. 2-inch paint brush.
4. Thickness gauge ¼-inch (a piece of ¼-inch plywood 2′ x 1″ is satisfactory).

C. Test Procedure

1. Determine and record grade with special level, see Fig. IV.

a. Place level on pavement parallel to direction of travel with adjustable end down grade.

b. Loosen locking screw and raise level until bubble centers and then tighten locking screw on sliding bar.

c. The grade is indicated on the calibrated sliding bar.

2. Remove apparatus from vehicle and attach to bumper hitch, Fig. V.

3. Position apparatus with tire over selected test area and parallel to direction of traffic.

4. Raise tire and adjust to ¼-inch (1⁄16″ tolerance) above surface to be tested with device (F).

5. Wet full circumference of tire and pavement surface under tire and 16″ ahead of tire center with glycerine, using a paint brush.

6. Set sliding gauge indicator (P) against carriage end.

7. Depress starting switch (J) and bring the speed to approximately 55 mi/hr.

8. Release starting switch.

9. The instant the tachometer shows 50 mi/hr trip arm (G) dropping tire to pavement.

10. Read gauge (N) and record.

11. Release rebound shock absorber.

12. Move to next section and repeat.

13. In any one test location, test at 25′ intervals in a longitudinal direction over a 100′ section of pavement.

D. Precautions

1. The rear support rod (O), Fig. II, must be cleaned by washing frequently with water and a detergent to prevent sticking.

2. Sliding gauge indicator (P) must be kept clean so that it will slide very freely.

3. On slick pavements glycerine remaining on the pavement should be flushed off with water to prevent possible traffic accidents.

E. Field Construction Testing of Portland Cement Concrete Pavement

The following procedure shall be followed in the field testing of a portland cement concrete pavement for specification compliance of the minimum friction value. A minimum of seven days after paving shall lapse before testing.

1. Visually survey the total length of pavement for uniformity of surface texture. Note all areas which do not have definite striations or which appear smooth. Conduct this survey with the Resident Engineer or an Assistant who has knowledge of any difficulties in attaining a proper surface texture during construction. The attached photograph, Figure VIII, may be used as an aid in the evaluation of the existing texture in relation to the coefficient of friction, but is not to be used in lieu of actual coefficient of friction measurements.

2. The determination of test locations, as outlined below, *shall apply only to that portion of the pavement which has well formed striations. All areas that appear smooth, or those that have been ground shall be excluded.* (See E-3 for procedure to follow for smooth pavements).

a. Select a minimum of three test locations for each day's pour and check a minimum of three pour days per contract.

Determine the location of test sites in a random manner through use of a Random Number table. The use of this method requires that the area for test be uniformly textured and placed in one operation. As an example, a 4-lane pavement may be placed with a three lane width in one operation and the fourth lane placed separately. Each of these areas must be treated separately in selecting test locations. The following example illustrates the use of this table.

A section of pavement is 24′ wide and 4000′ long and is part of a 4-lane freeway. This section of pavement has been placed in one operation and skid tests are required. From 2-a, it is required that three test locations be determined.

Using the random numbers, as shown, choose the three locations in the following manner:

Random Numbers *Longitudinal*	*Lateral*
0.6	6
0.9	9
0.2	2
0.7	7
0.5	5
0.1	11
0.4	4
0.8	8
0.3	3

Starting at any point and proceeding up, or down, but not skipping any numbers, read three pairs of numbers and set up each location as follows:

	Distance from Start of Pour	*Distance fromRight Edge of Pour Looking up Station*
Location A	$0.6 \times 4{,}000' = 2{,}400'$	$6 \times 2 = 12'$
Location B	$0.9 \times 4{,}000' = 3{,}600'$	$9 \times 2 = 18'$
Location C	$0.2 \times 4{,}000' = 800'$	$2 \times 2 = 4'$

In case any location as determined above falls in a smooth or ground area which does not appear representative of the general surface texture, then choose the next number in the random table and select a new location.

At each test location obtain the first reading at the specified random location (using the method described under C-Test Procedure). Obtain the next four readings at 25′ intervals beyond the first reading. Obtain all readings at sites parallel to the centerline of the lane. After correction for grade as shown in F, average the five readings. Record this average as the friction value for the specific test location.

3. In all areas that present a smooth textured appearance or have been ground, the following shall apply:

a. Check a minimum of three ground area locations and all smooth appearing surfaces on each contract.

b. If the area is less than 100′ in length perform at least three individual tests in separate spots, correct for grade and average the results.

c. If the area is greater than 100′ in length, select sufficient test locations to insure that the area is above the minimum requirement. If the average value of all locations is below the required minimum then perform additional tests until the area is localized for remedial action.

F. Calculations

1. Make grade corrections using charts shown in Figures VI and VII.

2. Average the 5 corrected readings in any one test location. *Example*—The following readings were taken at 25′ intervals in a test location. The grade of the pavement, determined as described in C-1, was +4%.

Station	*Measured Coefficient of Friction*	*Corrected Coefficient of Friction**
1+00	0.33	0.38
1+25	0.34	0.39
1+50	0.34	0.39
1+75	0.33	0.38
2+00	0.33	0.38
Final Average for Test Site		0.38

* Corrected coefficients of friction were taken from chart in Figure VI.

G. Reporting of Results

For all results determined under E-2, report the result for each station location and the average of 5 readings and the grand average. For all results determined under E-3, part (b), report the result for each station location and the average. For E-3, part (c), report the result for each station location and the average for each set of five determinations.

REFERENCE

A California Method

End of Text on Calif. 342-C

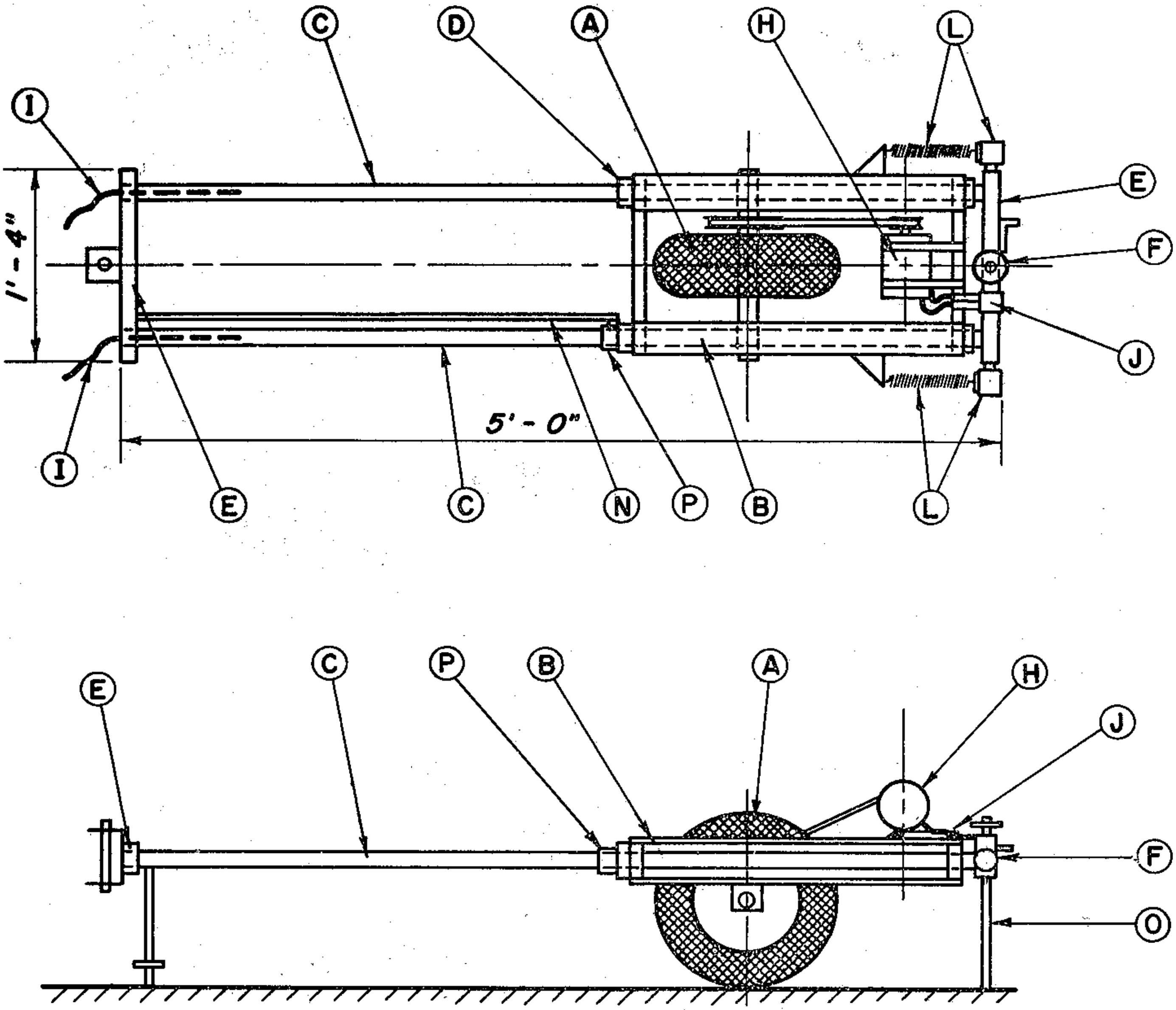


FIGURE I
DIAGRAM OF SKID TESTER

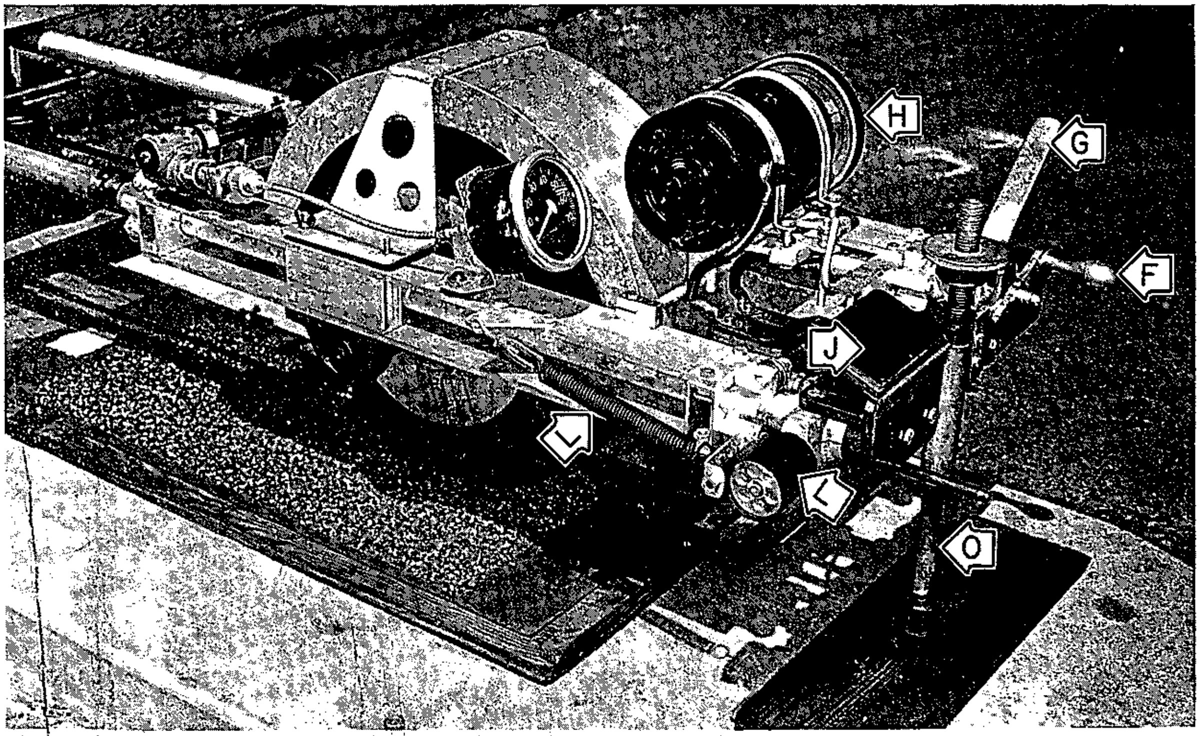


FIGURE II

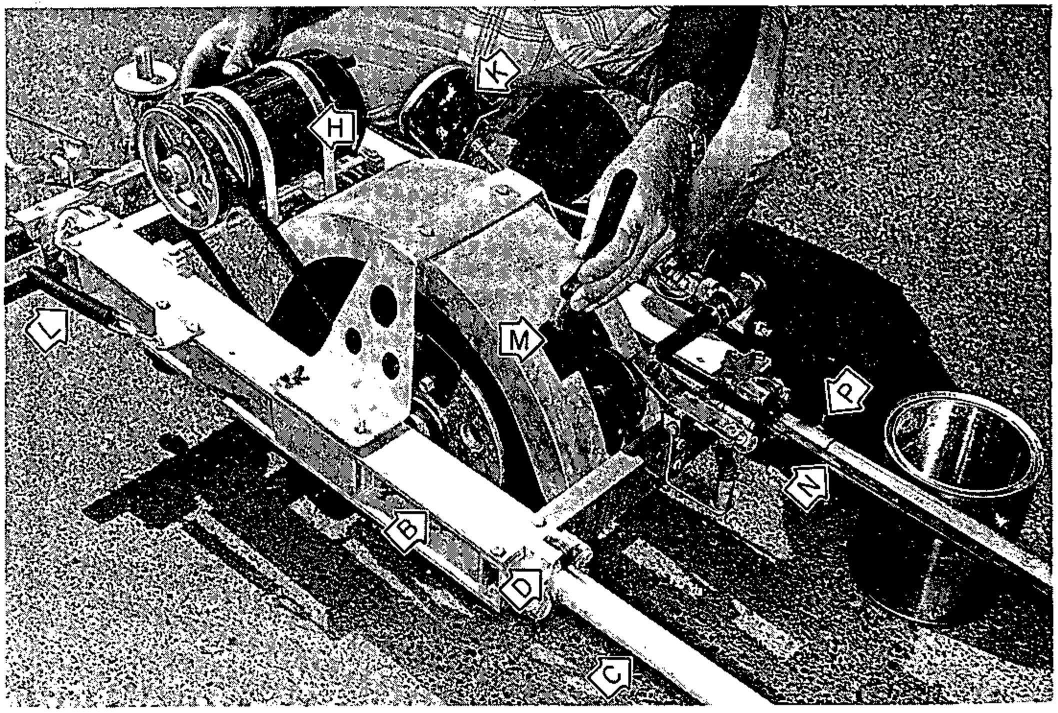


FIGURE III
CLOSE-UP VIEWS OF SKID TESTER

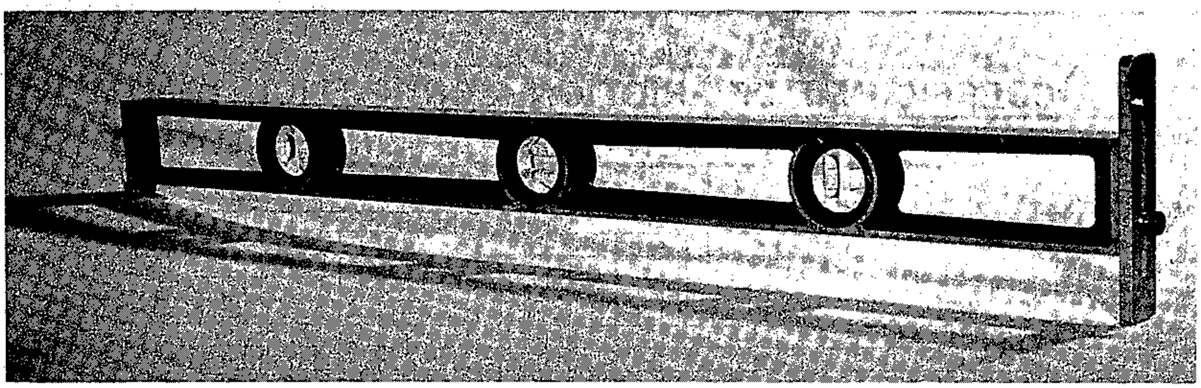


FIGURE IV
LEVEL FOR DETERMINING GRADE

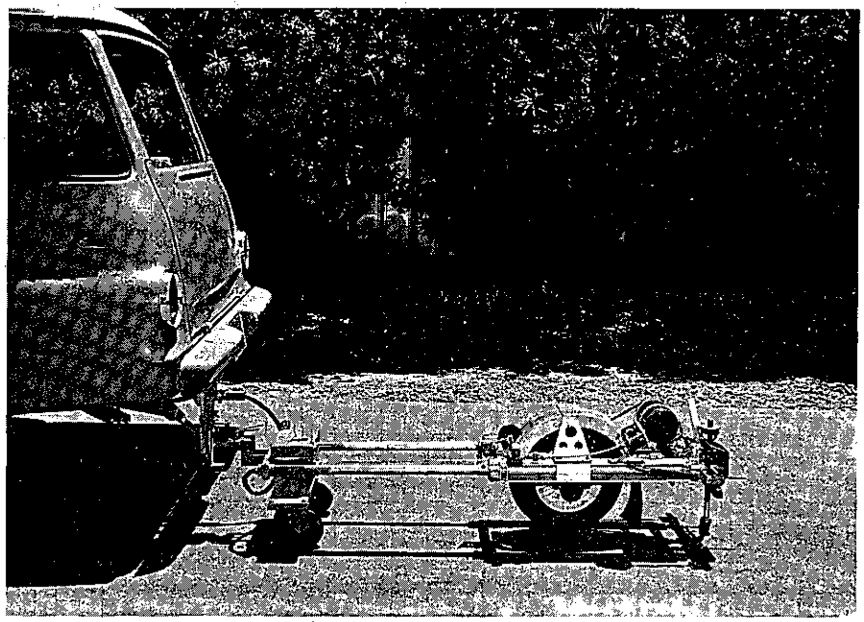


FIGURE V
APPARATUS IN POSITION FOR TESTING

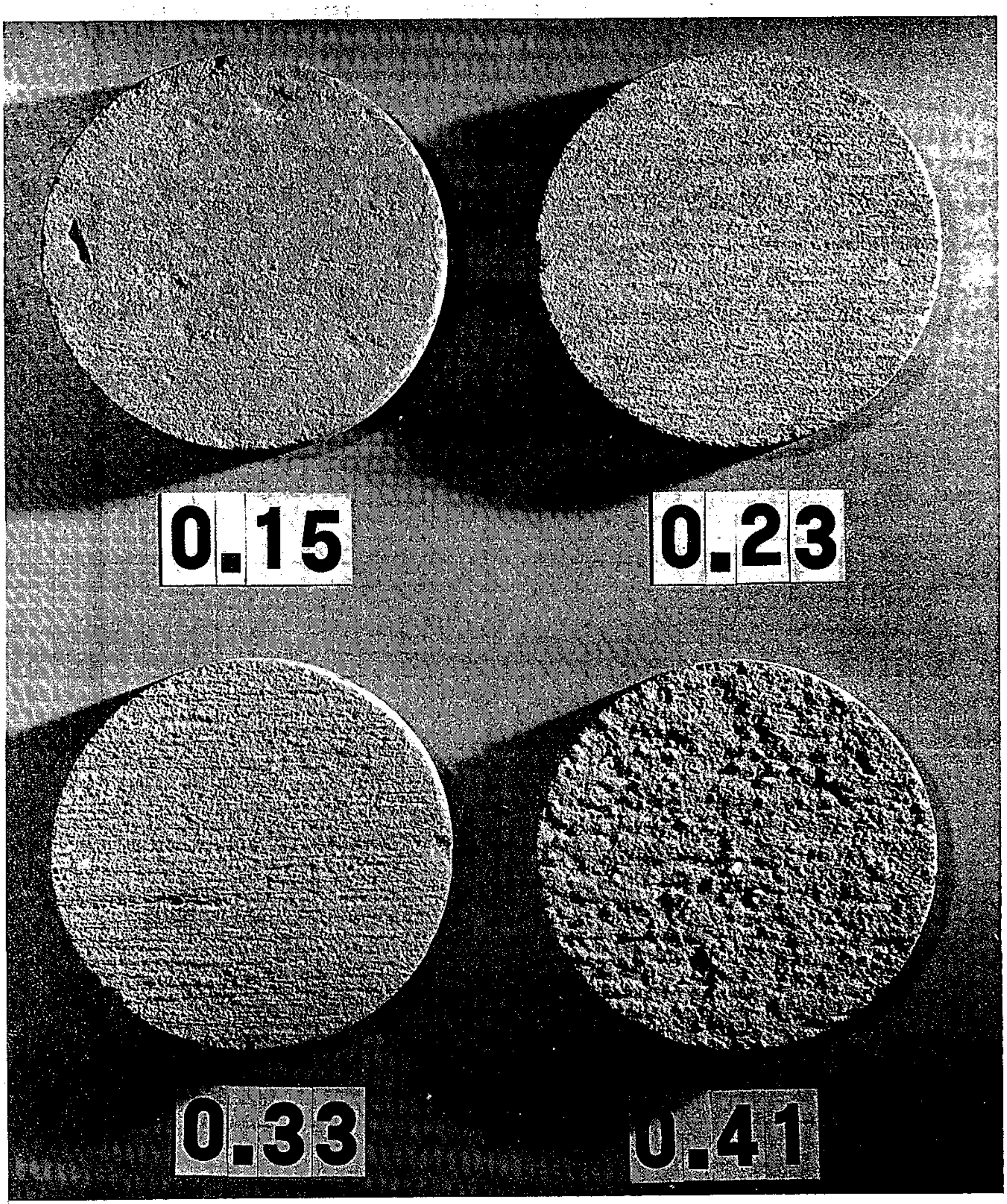


FIGURE VIII
PHOTOS OF SURFACE TEXTURES

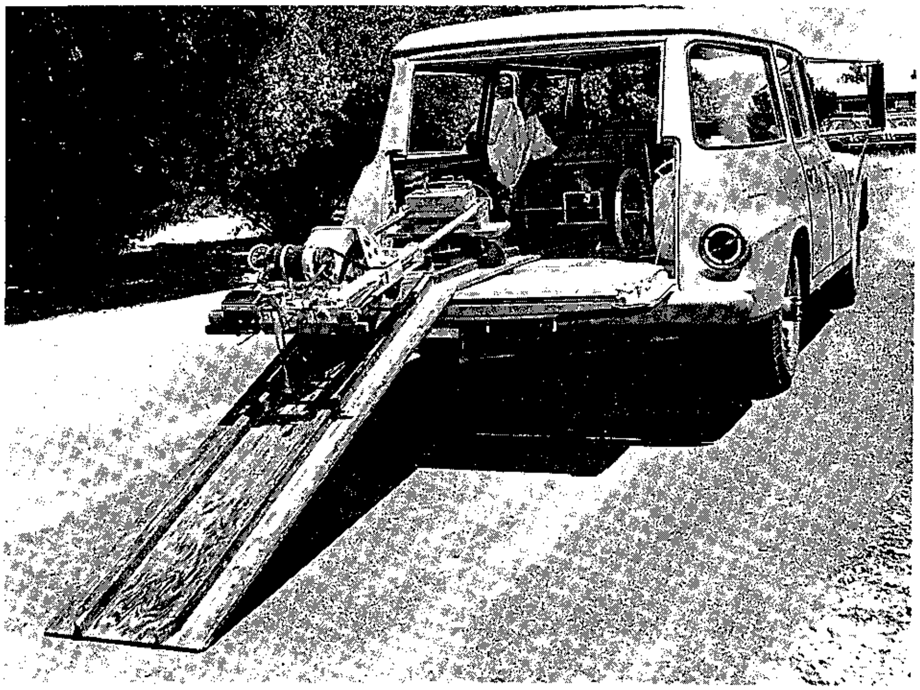


FIGURE IX

APPARATUS BEING PLACED IN VEHICLE
NOTE CABLE AND WINCH FOR MOVING SKID TESTER

FIGURE X

APPARATUS IN POSITION FOR TRANSPORTATION